DIVISION 6
STRUCTURES

6-01 GENERAL REQUIREMENTS FOR STRUCTURES

6-01.1 Description

This section relates to structural and incidental items used in any or all types of existing or proposed structures. These provisions supplement the detailed specifications supplied for any given structure. These provisions apply only when relevant and when they do not conflict with the Plans or Special Provisions.

6-01.2 Foundation Data

Foundation data in the Plans (from test borings, test pits, or other sources) were obtained only to guide the Department in planning and designing the project. These data reasonably represent the best information available to the Department concerning conditions and materials at the test sites at the time the investigations were made.

6-01.3 Clearing the Site

The Contractor shall clear the entire site of the proposed structure to the limits staked by the Engineer.

6-01.4 Appearance of Structures

To achieve a more pleasing appearance, the Engineer may require the Contractor to adjust the height and alignment of bridge railings, traffic barrier, and structural curbs.

6-01.5 Vacant

6-01.6 Load Restrictions on Bridges Under Construction

Bridges under construction shall remain closed to all traffic, including construction equipment, until the substructure and the superstructure, through the roadway deck, are complete for the entire structure, except as provided herein. Completion includes release of all falsework, removal of all forms, and attainment of the minimum design concrete strength and specified age of the concrete in accordance with these Specifications. Once the structure is complete, Section-1-07.7 shall govern all traffic loading, including construction traffic (equipment).

If necessary and safe to do so, and if the Contractor requests it in writing, the Engineer may approve traffic on a bridge prior to completion. The maximum distributed load at each construction equipment support shall not exceed the design load by more than 33 percent. The written request shall:

1. Describe the extent of the structure completion at time of the proposed equipment loading;
2. Describe the loading magnitude, arrangement, movement, and position of traffic (equipment) on the bridge, including but not limited to the following:
    a. Location of construction equipment, including outriggers, spreader beams and supports for each, relative to the bridge framing plan (bridge girder layout);
    b. Mechanism of all load transfer (load path) to the bridge;

3. Provide stress calculations under the design criteria specified in the AASHTO Standard Specifications for Highway Bridges, current edition, prepared by (or under the direction of) a professional engineer, licensed under Title 18 RCW state of Washington, and carrying the professional engineer's signature and seal, including but not limited to the following:
    a. Supporting calculations showing that the flexural and shear stresses in the main load carrying members due to the construction load are within the allowable stresses;
    b Supporting calculations showing that the flexural and shear stresses in the bridge deck due to the construction load are within the allowable stresses;
4. Provide supporting material properties, catalogue cuts, and other information describing the construction equipment and all associated outriggers, spreader beams, and supports; and,
5. State that the Contractor assumes all risk for damage.

6-01.7 Navigable Streams

The Contractor shall keep navigable streams clear so that water traffic may pass safely, providing and maintaining all lights and signals required by the U.S. Coast Guard. The Contractor shall also comply with all channel depth and clearance line requirements of the U.S. Corps of Engineers. This may require removing material deposited in the channel during construction.

6-01.8 Approaches to Movable Spans

No roadway or sidewalk slab on the approach span at either end of a movable span may be placed until after the movable span has been completed, adjusted and closed.

6-01.9 Working Drawings

The Contractor shall submit supplemental working drawings with calculations as required for the performance of the work. The drawings shall be on sheets measuring 22 by 34-inches, 11 by 17-inches, or on sheets with dimensions in multiples of $8^1/_2$ by 11-inches. All drawings shall be to scale in keeping with standard drafting procedures. The design calculations shall be on sheets measuring $8^1/_2$ by 11-inches. They shall be legible, with all terms identified, and may include computer printouts. The drawings and calculations shall be provided far enough in advance of actual need to allow for the review process by the Contracting Agency, which may involve rejection, revision, or resubmittal. Unless otherwise stated in the contract, the Engineer will require up to 30 calendar days from the date the submittals are received until they are sent to the Contractor. This time will increase if the drawings submitted do not meet the contract requirements or contain insufficient details.

Unless designated otherwise by the Contractor, submittals of working drawing plans will be reviewed in the order they are received by the Engineer. In the event that several working drawing plans are submitted simultaneously, the Contractor shall specify the sequence in which these plans are to be reviewed. The Engineer's review time shall be as specified above for the first plan in the specified sequence and up to an additional two weeks for each plan lower in the specified sequence. A plan is defined as one or more working drawings that pertain to a unit of superstructure or a complete pier. If the Contractor does not submit a working drawing review sequence for simultaneous plan submittals, the review sequence shall be at the Engineer's discretion.

Working drawings and calculations shall be prepared by (or under the direction of) a Professional Engineer, licensed under Title 18 RCW, State of Washington, and shall carry the Professional Engineer's signature and seal.

If more than the specified number of days are required for the Engineer's review of any individual submittal or resubmittal, an extension of time will be considered in accordance with Section 1-08.8.

6-01.10 Utilities Supported by or Attached to Bridges

Installation of utility pipes and conduit systems shall conform to the details shown in the Plans and as specified in the utility agreement between the utility company and the Contracting Agency.

All utility pipes and conduit systems supported by or attached to bridge structures shall be labeled with Type I reflective sheeting conforming to Section 9-28.12, and the following:

Content	Label Background Color	Lettering Utility Color
Electrical Power	Red	Black
Gas, Oil, Steam, Petroleum, and other gaseous materials	Yellow	Black
CATV, Telecommunication, Alarm, and Signal	Orange	Black
Potable Water	Blue	White
Reclaimed Water,Irrigation, Slurry	Purple	White
Sewer and Storm Drain	Green	White

The purple color background for the label for reclaimed water, irrigation, and slurry, shall be generated by placing transparent film over white reflective material. The purple tint of the transparent film shall match Federal Standard Color 595B No. 37100. Color chips are available from the source specified in Section 9-08.4(7).

The label text shall identify the utility contents and include the emergency one-call phone number 1-800-424-5555.

The minimum length of the label color field shall be the longer of either one letter width beyond each end of the label text, or the length specified below:

Minimum Pipe O.D. (inches)	Maximum Pipe O.D. (inches)	Minimum Length of Label Color Field (inches)	Letter Height (inches)
$^3/_4$	$1^1/_4$	8	$^1/_2$
$1^1/_2$	2	8	$^3/_4$
$2^1/_2$	6	12	$1^1/_4$
8	10	24	$2^1/_2$
12	—	32	$3^1/_2$

Utility pipes and conduit systems shall be labeled on both sides of each bridge pier, and adjacent to each entrance hatch into a box girder cell. For utility pipes and conduit systems within bridge spans exceeding 300-feet, labels shall also be applied to the utility pipes and conduit systems between the piers at a maximum spacing of 300-feet. The label shall be visible at a normal eye height.

6-01.11 Name Plates

The Contractor shall install no permanent plates or markers on a structure unless the Plans show it.

6-01.12 Final Cleanup

When the structure is completed, the Contractor shall leave it and the entire site in a clean and orderly condition. Structure decks shall be swept and washed. Temporary buildings, falsework, piling, lumber, equipment, and debris shall be removed. The Contractor shall level and fine grade all excavated material not used for backfill, and shall fine grade all slopes and around all piers, bents, and abutments.

The Contractor is advised that after the structure is complete, a representative(s) of the WSDOT Bridge Preservation Office may perform an Inventory Inspection of the structure. The purpose of the Inventory Inspection is to field verify certain contract details, to provide a base-line condition assessment of the structure, and to identify any potential maintenance features.

6-01.13 Architectural Features

To ensure uniform texture and color, the Contractor shall obtain all cement for the structure from the same manufacturing plant unless the Engineer waives this requirement in writing.

6-01.14 Premolded Joint Filler

When the Plans call for premolded joint filler, the Contractor shall fasten it with galvanized wire nails to one side of the joint. The nails must be no more than 6-inches apart and shall be $1^1/_2$-inches from the edges over the entire joint area. The nails shall be at least $1^1/_2$-inches longer than the thickness of the filler.

The Contractor may substitute for the nails any adhesive approved by the Engineer. This adhesive, however, shall be compatible with Resilient Bituminous Preformed Expansion Joint Filler (ASTM D 1751) and capable of bonding the filler to Portland cement concrete.

6-01.15 Normal Temperature

Bridge plans state dimensions at a normal temperature of 64ºF. Unless otherwise noted, these dimensions are horizontal or vertical.

6-02 CONCRETE STRUCTURES

6-02.1 Description

This work consists of the construction of all structures (and their parts) made of Portland cement concrete with or without reinforcement. Any part of a structure to be made of other materials shall be built as these Specifications require elsewhere.

6-02.2 Materials

Materials shall meet the requirements of the following sections:

Portland Cement	9-01
Aggregates for Portland Cement Concrete	9-03.1
Gravel Backfill	9-03.12
Joint and Crack Sealing Materials	9-04
Reinforcing Steel	9-07
Epoxy-Coated Reinforcing Steel	9-07
Prestressed Concrete Girders	9-19
Curing Materials and Admixtures	9-23
Fly Ash	9-23
Microsilica Fume	9-23.11
Plastic Waterstop	9-24
Water	9-25
Elastomeric Bearing Pads	9-31

6-02.3 Construction Requirements

6-02.3(1) Classification of Structural Concrete

The class of concrete to be used shall be as noted in the Plans and these Specifications. The numerical class of concrete defines the specified minimum compressive strength at 28 days in accordance with AASHTO T 22. The letter designation following the class of concrete identifies the specific use; P for Piling applications, W for Underwater applications, and D for Deck applications.

The Contractor may request, in writing, permission to use a different class of concrete with either the same or a higher compressive strength than specified. The substitute concrete shall be evaluated for acceptance based on the specified class of concrete. The Engineer will respond in writing. The Contractor shall bear any added costs that result from the change.

6-02.3(2) Proportioning Materials

The total water soluble Chloride ion (Cl-) content of the mixed concrete shall not exceed 0.06 percent by weight of cementitious material for prestressed concrete nor 0.10 percent by weight of cementitious material for reinforced concrete. An initial evaluation may be obtained by testing individual concrete ingredients for total chloride ion content per AASHTO T 260 and totaling these to determine the total water soluble Chloride ion (Cl-) or the total water soluble Chloride ion (Cl-) in accordance with ASTM C 1218.

Unless otherwise specified, the Contractor shall use Type I or II Portland cement in all concrete as defined in Section 9-01.2(1).

The use of fly ash is required for Class 4000D and 4000P concrete. The use of fly ash and ground granulated blast furnace slag is optional for all other classes of concrete.

Fly ash, if used, shall not exceed 35 percent by weight of the total cementitious material and shall conform to Section 9-23.9. Ground granulated blast furnace slag, if used, shall not exceed 25 percent by weight of the total cementitious material and shall conform to Section 9-23.10. When both ground granulated blast furnace slag and fly ash are included in the concrete mix, the total weight of both these materials is limited to 35 percent by weight of the total cementitious material.

The water/cement ratio shall be calculated on the total weight of cementitious material. The following are considered cementitious materials: Portland cement, fly ash, ground granulated blast furnace slag and microsilica.

As an alternative to the use of fly ash, ground granulated blast furnace slag and cement as separate components, a blended hydraulic cement that meets the requirements of Section 9-01.2(4) Blended Hydraulic Cements may be used.

6-02.3(2)A Contractor Mix Design

The Contractor shall provide a mix design in writing to the Engineer for all classes of concrete specified in the Plans except for those accepted based on a Certificate of Compliance. No concrete shall be placed until the Engineer has reviewed the mix design. The required average 28 day compressive strength shall be selected per ACI 318, chapter 5, Section 5.3.2. ACI 211.1 and ACI 318 shall be used to determine proportions. The proposed mix for Class 4000P shall provide a minimum fly ash content per cubic yard of 100 pounds and a minimum cement content per cubic yard of 600 pounds. The proposed mix for Class 4000D shall provide a minimum fly ash content per cubic yard of 75 pounds and a minimum cement content per cubic yard of 660 pounds. All other concrete mix designs, except those for lean concrete and commercial concrete, shall have a minimum cementitious material content of 564 pounds per cubic yard of concrete.

The Contractor's submittal of a mix design shall be on WSDOT form 350-040 and shall provide a unique identification for each mix design and shall include the mix proportions per cubic yard, the proposed sources, the average 28 day compressive strength for which the mix is designed, the fineness modulus, water cement ratio, and the aggregate correction factor per WAQTC FOP for AASHTO T 152. Concrete placeability, workability, and strength shall be the responsibility of the Contractor. The Contractor shall notify the Engineer in writing of any mix design modifications.

Fine aggregate shall conform to Section 9-03.1(2) Class 1 or Class 2.

Coarse aggregate shall conform to Section 9-03. The nominal maximum size aggregate for Class 4000P shall be 1/2-inch. The nominal maximum size aggregate for Class 4000D shall be 3/4-inch.

Nominal maximum size for concrete aggregate is defined as the smallest standard sieve opening through which the entire amount of the aggregate is permitted to pass.

Class 4000D and 4000P concrete shall include a water reducing admixture in the amount recommended by the manufacturer. A retarding admixture is required in concrete Class 4000P. Water reducing and retarding admixtures are optional for all other concrete classes.

A high-range water reducer (superplasticizer) may be used in all mix designs. Microsilica fume may be used in all mix designs. The use of a high-range water reducer or microsilica fume shall be submitted as a part of the Contractor's concrete mix design.

Air content shall be a minimum of 4.5 percent and a maximum of 7.5 percent for all concrete placed above the finished ground line.

6-02.3(2)B Commercial Concrete

Commercial concrete shall have a minimum compressive strength at 28 days of 3000 psi in accordance with WSDOT FOP for AASHTO T 22. Commercial concrete placed above the finished ground line shall be air entrained and have an air content from 4.5 percent to 7.5 percent per WAQTC FOP for AASHTO T 152. Commercial concrete does not require plant approval, mix design, or source approvals for cement, aggregate, and other admixtures.

Where concrete Class 3000 is specified for nonstructural items such as, culvert headwalls, plugging culverts, concrete pipe collars, pipe anchors, monument cases, luminaire bases, pedestals, cabinet bases, guardrail anchors, sign post foundations, fence post footings, sidewalks, curbs, and gutters, the Contractor may use commercial concrete. If commercial concrete is used for sidewalks, curbs, and gutters, it shall have a minimum cementitious material content of 564 pounds per cubic yard of concrete, shall be air entrained, and the tolerances of Section 6-02.3(5)C shall apply. Commercial concrete shall not be used for structural items such as, bridges, retaining walls, box culverts, or foundations for high mast luminaires, mast arm traffic signals, cantilever signs, and sign bridges. The Engineer may approve other nonstructural items not listed for use as commercial concrete.

6-02.3(2)C Vacant

6-02.3(2)D Lean Concrete

Lean concrete shall contain between 145 and 200 pounds of cement per cubic yard and have a maximum water/cement ratio of 2.

6-02.3(3) Admixtures

Concrete admixtures shall be added to the concrete mix at the time of batching the concrete or in accordance with the manufacturer's written procedure and as approved by the Engineer. A copy of the manufacturer's written procedure shall be furnished to the Engineer prior to use of any admixture. Any deviations from the manufacturer's written procedures shall be submitted to the Engineer for approval. Admixtures shall not be added to the concrete with the modified procedures until the Engineer has approved them in writing.

When the Contractor is proposing to use admixtures from different admixture manufacturers they shall provide evidence to the Engineer that the admixture will be compatible and not adversely effect the air void system of the hardened concrete. Test results complying with ASTM C 457 shall be provided as the evidence to satisfy this requirement. Admixture combinations which have been previously tested and which are in compliance with ASTM C 457 shall be listed in the Qualified Products List (QPL). Proposed combinations not found in the QPL shall meet this requirement.

Accelerators shall not be used.

Air entrained cement shall not be used to air entrain concrete.

6-02.3(4) Ready-Mix Concrete

All concrete, except commercial concrete and lean concrete shall be batched in a prequalified manual, semi-automatic, or automatic plant as described in Section 6-02.3(4)A. The Engineer is not responsible for any delays to the Contractor due to problems in getting the plant certified.

6-02.3(4)A Qualification of Concrete Suppliers

Prequalification may be obtained through an inspection conducted by the Plant Manager, defined as the person directly responsible for the daily plant operation, using the NRMCA or WSDOT checklist, through certification by NRMCA, or by an independent evaluation certified by a professional engineer using NRMCA or Contracting Agency guidelines. Information concerning NRMCA certification may be obtained from the National Ready Mix Concrete Association at 900 Spring Street, Silver Springs, MD 20910. The Contracting Agency and the NRMCA certification have similar requirements for plant and delivery equipment. Whereas Plant Manager certification shall be done prior to the start of a project and every six months throughout the life of the project, the NRMCA certification shall be good for a two year period.

If prequalification is done by the Plant Manager the following shall be performed:

1. The checklist cover page shall be signed by the Plant Manager and notarized.
2. The signed and notarized cover page shall be submitted to the Project Engineer with the concrete mix design (WSDOT Form 350-040), water meter verification, truck list, and admixture dispensing certification.
3. The checklists shall be maintained by the Plant Manager and are subject to review at any time by the Contracting Agency.
4. The water meter shall be verified every six months.

For central-mixed concrete, the mixer shall be equipped with a timer that prevents the batch from discharging until the batch has been mixed for the prescribed mixing time. A mixing time of one minute will be required after all materials and water have been introduced into the drum. Shorter mixing time may be allowed if the mixer performance is tested in accordance with (AASHTO M 157 Annex A1 Concrete Uniformity Requirements). Tests shall be conducted by an independent testing lab or by a commercial concrete producer's lab. If the tests are performed by a producer's lab, the Engineer or a representative will witness all testing.

For shrink-mixed concrete, the mixing time in the stationary mixer shall not be less than 30 seconds or until the ingredients have been thoroughly blended.

For transit-mixed or shrink-mixed concrete, the mixing time in the transit mixer shall be a minimum of 70 revolutions at the mixing speed designated by the manufacturer of the mixer. Following mixing, the concrete in the transit mixer may be agitated at the manufacturer's designated agitation speed. A maximum of 320 revolutions (total of mixing and agitation) will be permitted prior to discharge.

All transit-mixers shall be equipped with an operational revolution counter and a functional device for measurement of water added. All mixing drums shall be free of concrete buildup and the mixing blades shall meet the minimum specifications of the drum manufacturer. A copy of the manufacturer's blade dimensions and configuration shall be on file at the concrete producer's office. A clearly visible metal data plate (or plates) attached to each mixer and agitator shall display: (1) the maximum concrete capacity of the drum or container for mixing and agitating, and (2) the rotation speed of the drum or blades for both the agitation and mixing speeds. Mixers and agitators shall always operate within the capacity and speed-of-rotation limits set by the manufacturer. Any mixer, when fully loaded, shall keep the concrete uniformly mixed. All mixers and agitators shall be capable of discharging the concrete at a steady rate. Only those transit-mixers which meet the above requirements will be allowed to deliver concrete to any Contracting Agency project covered by these Specifications.

In transit-mixing, mixing shall begin within 30 seconds after the cement is added to the aggregates.

Central-mixed concrete, transported by truck mixer/agitator, shall not undergo more than 250 revolutions of the drum or blades before beginning discharging. To remain below this limit, the suppler may agitate the concrete intermittently within the prescribed time limit. When water or admixtures are added after the load is initially mixed, an additional 30 revolutions will be required at the recommended mixing speed.

For each project, at least biannually, or as required, the Plant Manager will examine mixers and agitators to check for any buildup of hardened concrete or worn blades. If this examination reveals a problem, or if the Engineer wishes to test the quality of the concrete, slump tests may be performed with samples taken at approximately the $^1/_4$ and $^3/_4$ points as the batch is discharged. The maximum allowable slump difference shall be as follows:

If the average of the two slump tests is $\leq$ 4-inches, the difference shall be $\leq$ 1-inch or if the average of the two slump tests is >4-inches, the difference shall be $\leq 1^1/_2$-inches.

If the slump difference exceeds these limits, the equipment shall not be used until the faulty condition is corrected. However, the equipment may continue in use if longer mixing times or smaller loads produce batches that pass the slump uniformity tests.

All concrete production facilities will be subject to verification inspections at the discretion of the Engineer. Verification inspections are a check for: current scale certifications; accuracy of water metering devices; accuracy of the batching process; and verification of coarse aggregate quality.

If the concrete producer fails to pass the verification inspection, the following actions will be taken:

1. For the first violation, a written warning will be provided.
2. For the second violation, the Engineer will give written notification and the Contracting Agency will assess a price reduction equal to 15 percent of the invoice cost of the concrete that is supplied from the time of the infraction until the deficient condition is corrected.
3. For the third violation, the concrete supplier is suspended from providing concrete until all such deficiencies causing the violation have been permanently corrected and the plant and equipment have been reinspected and meets all the prequalification requirements.
4. For the fourth violation, the concrete supplier shall be disqualified from supplying concrete for one year from the date of disqualification. At the end of the suspension period the concrete supplier may request that the facilities be inspected for prequalification.

6-02.3(4)B Jobsite Mixing

For small quantities of concrete, the Contractor may mix concrete on the job site provided the Contractor has requested in writing and received written permission from the Engineer. The Contractor's written request shall include a mix design, batching and mixing procedures, and a list of the equipment performing the job-site mixing. All job site mixed concrete shall be mixed in a mechanical mixer.

If the Engineer permits, hand mixing of concrete will be permitted for pipe collars, pipe plugs, fence posts, or other items approved by the Engineer, provided the hand mixing is done on a watertight platform in a way that distributes materials evenly throughout the mass. Mixing shall continue long enough to produce a uniform mixture. No hand mixed batch shall exceed $^1/_2$ cubic yard.

Concrete mixed at the jobsite is never permitted for placement in water.

6-02.3(4)C Consistency

The maximum slump for concrete shall be:

1. 3.5-inches for vibrated concrete placed in all bridge roadway slabs, bridge approach slabs, and flat slab bridge superstructures.
2. 4.5-inches for all other vibrated concrete.
3. 7-inches for non-vibrated concrete. (Includes Class 4000P)
4. 9-inches for shafts when using Class 4000P, provided the water cement ratio does not exceed 0.44 and a water reducer is used meeting the requirements of 9-23.6.
5. 5.5-inches for all concrete placed in curbs, gutters, and sidewalks.

When a high range water reducer is used, the maximum slump listed in 1, 2, 3, and 5 above, may be increased an additional 2-inches.

6-02.3(4)D Temperature and Time For Placement

Concrete temperatures shall remain between 55ºF and 90ºF while it is being placed. Precast concrete that is heat cured per Section 6-02.3(25)D shall remain between 50ºF and 90ºF while being placed. The batch of concrete shall be discharged at the project site no more than $1^1/_2$ hours after the cement is added to the concrete mixture. The time to discharge may be extended to $1^3/_4$ hours if the temperature of the concrete being placed is less than 75ºF. With the approval of the Engineer and as long as the temperature of the concrete being placed is below 75ºF, the maximum time to discharge may be extended to two hours. When conditions are such that the concrete may experience an accelerated initial set, the Engineer may require a shorter time to discharge. The time to discharge may be extended upon written request from the Contractor. This time extension will be considered on a case by case basis and requires the use of specific retardation admixtures and the approval of the Engineer.

6-02.3(5) Acceptance of Concrete

6-02.3(5)A General

Lean concrete and commercial concrete will be accepted based on a Certificate of Compliance to be provided by the supplier as described in Section 6-02.3(5)B.

All other concrete will be accepted based on conformance to the requirement for temperature, slump, air content for concrete placed above finished ground line, and the specified compressive strength at 28 days for sublots as tested and determined by the Contracting Agency.

A sublot is defined as the material represented by an individual strength test. An individual strength test is the average compressive strength of cylinders from the same sample of material.

Each sublot will be deemed to have met the specified compressive strength requirement when both of the following conditions are met:

1. Individual strength tests do not fall below the specified strength by more than 12½ percent or 500 psi, whichever is least.
2. An individual strength test averaged with the two preceding individual strength tests meets or exceeds specified strength (for the same class and exact mix I.D. of concrete on the same contract).

When compressive strengths fail to satisfy one or both of the above requirements, the Contractor may:

1. Request acceptance based on the Contractor/Suppliers strength test data for cylinders made from the same truckload of concrete as the Contracting Agency cylinders; provided:
   a. The Contractor's test results are obtained from testing cylinders fabricated, handled, and stored for 28 days in accordance with WSDOT FOP for AASHTO T 23 and tested in accordance with AASHTO T 22. The test cylinders shall be the same size cylinders as those cast by the Contracting Agency.
   b. The technician fabricating the cylinders is qualified by either ACI, Grade 1 or WAQTC to perform this work.
   c. The laboratory performing the tests per AASHTO T 22 has an equipment calibration/certification system, and a technician training and evaluation process per AASHTO R-18.
   d. Both the Contractor and Contracting Agency have at least 15 test results from the same mix to compare. The Contractor's results could be used if the Contractor's computed average of all their test results is within one standard deviation of the Contracting Agency's average test result. The computed standard deviation of the Contractor's results must also be within plus or minus 200 psi of the Contracting Agency's standard deviation.
2. Request acceptance of in-place concrete strength based on core results. This method will not be used if the Engineer determines coring would be harmful to the integrity of the structure. Cores, if allowed, will be obtained by the Contractor in accordance with AASHTO T 24 and delivered to the Contracting Agency for testing in accordance with AASHTO T 22. If the concrete in the structure will be dry under service conditions, the core will be air dried at a temperature of between 60ºF and 80ºF and at a relative humidity of less than 60 percent for seven days before testing, and will be tested air dry.

Acceptance for each sublot by the core method requires that the average compressive strength of three cores be at least 85 percent of the specified strength with no one core less than 75 percent of the specified strength. When the Contractor requests strength analysis by coring, the results obtained will be accepted by both parties as conclusive and supersede all other strength data for the concrete sublot.

If the Contractor elects to core, cores shall be obtained no later than 50 days after initial concrete placement. The Engineer will concur in the locations to be cored. Repair of cored areas shall be the responsibility of the Contractor. The cost incurred in coring and testing these cores, including repair of core locations, shall be borne by the Contractor.

6-02.3(5)B Certification of Compliance

The concrete producer shall provide a Certificate of Compliance for each truckload of concrete. The Certificate of Compliance shall verify that the delivered concrete is in compliance with the mix design and shall include:

Manufacturer plant (batching facility)

Contracting Agency contract number.

Date

Time batched

Truck No.

Initial revolution counter reading

Quantity (quantity batched this load)

Type of concrete by class and producer design mix number

Cement producer, type, and Mill Certification No. (The mill test number as required by Section 9-01.3 is the basis for acceptance of cement.)

Fly ash (if used) brand and Type

Approved aggregate gradation designation

Mix design weight per cubic yard and actual batched weights for:

Cement

Fly ash (if used)

Coarse concrete aggregate and moisture content (each size)

Fine concrete aggregate and moisture content

Water (including free moisture in aggregates)

Admixtures brand and total quantity batched

Air-entraining admixture

Water reducing admixture

Other admixture

The Certificate of Compliance shall be signed by a responsible representative of the concrete producer, affirming the accuracy of the information provided. In lieu of providing a machine produced record containing all of the above information, the concrete producer may use the Contracting Agency-provided printed forms, which shall be completed for each load of concrete delivered to the project.

For commercial concrete, the Certificate of Compliance shall include, as a minimum, the batching facility, date, and quantity batched per load.

6-02.3(5)C Conformance to Mix Design

Cement, coarse and fine aggregate weights shall be within the following tolerances of the mix design:

Batch Volumes less than or equal to 4 cubic yards		
Cement	+5%	-1%
Aggregate	+10%	-2%
Batch Volumes more than 4 cubic yards		
Cement	+5%	-1%
Aggregate	+2%	-2%

If the total cementitious material weight is made up of different components, these component weights shall be within the following tolerances:

1. Portland cement weight plus 5% or minus 1 percent of that specified in the mix design.
2. Fly ash weight plus or minus 5 percent of that specified in the mix design.
3. Microsilica weight plus or minus 10 percent of that specified in the mix design.

Water shall not exceed the maximum water specified in the mix design.

6-02.3(5)D Test Methods

Acceptance testing will be performed by the Contracting Agency in accordance with the WSDOT Materials Manual. The test methods to be used with this specification are:

WSDOT FOP for AASHTO T 22	Compressive Strength of Cylindrical Concrete Specimens
WSDOT FOP for AASHTO T 23	Making and Curing Concrete Test Specimens in the Field
WSDOT FOP for AASHTO T 119	Slump of Hydraulic Cement Concrete
FOP for WAQTC TM 2	Sampling Freshly Mixed Concrete
WAQTC FOP for AASHTO T 152	Air Content of Freshly Mixed Concrete by the Pressure Method
WSDOT FOP for AASHTO T 231	Capping Cylindrical Concrete Specimens
WSDOT FOP for AASHTO T 309	Temperature of Freshly Mixed Portland Cement Concrete

6-02.3(5)E Point of Acceptance

Determination of concrete properties for acceptance will be made based on samples taken as follows:

Bridge decks, overlays, and barriers at the discharge of the placement system.

All other placements at the truck discharge.

It shall be the Contractor’s responsibility to provide adequate and representative samples of the fresh concrete to a location designated by the Engineer for the testing of concrete properties and making of cylinder specimens. Samples shall be provided as directed in Sections 1-06.1 and 1-06.2. Once the Contractor has turned over the concrete for acceptance testing, no more mix adjustment will be allowed. The concrete will either be accepted or rejected.

6-02.3(5)F Water/Cement Ratio Conformance

The actual water cement ratio shall be determined from the certified proportions of the mix, adjusting for on the job additions. No water may be added after acceptance testing or after placement has begun, except for concrete used in slip forming. For slip-formed concrete, water may be added during placement but shall not exceed the maximum water cement ratio in the mix design, and shall meet the requirements for consistency as described in Section 6-02.3(4)C. If water is added, an air and temperature test shall be taken prior to resuming placement to ensure that specification conformance has been maintained.

6-02.3(5)G Sampling and Testing Frequency for Temperature, Consistency, and Air Content

Concrete properties shall be determined from concrete as delivered to the project and as accepted by the Contractor for placement. The Contracting Agency will test for acceptance of concrete for slump, temperature, and air content, if applicable, as follows:

Sampling and testing will be performed before concrete placement from the first truck load. Concrete shall not be placed until tests for slump, temperature, and entrained air (if applicable) have been completed by the Engineer, and the results indicate that the concrete is within acceptable limits. Except for the first load of concrete, up to $1/2$ cubic yard may be placed prior to testing for acceptance. Sampling and testing will continue for each load until two successive loads meet all applicable acceptance test requirements. After two successive tests indicate that the concrete is within specified limits, the sampling and testing frequency may decrease to one for every five truck loads. Loads to be sampled will be selected in accordance with the random selection process as outlined in WAQTC FOP for TM 2.

When the results for any subsequent acceptance test indicates that the concrete as delivered and approved by the Contractor for placement does not conform to the specified limits, the sampling and testing frequency will be resumed for each truck load. Whenever two successive subsequent tests indicate that the concrete is within the specified limits, the random sampling and testing frequency of one for every five truck loads may resume.

Sampling and testing for a placement of one class of concrete consisting of 50 cubic yards or less will be as listed above, except:

Sampling and testing will continue until one load meets all of the applicable acceptance requirements, and

After one set of tests indicate that the concrete is within specified limits, the remaining concrete to be placed may be accepted by visual inspection.

6-02.3(5)H Sampling and Testing for Compressive Strength

Acceptance testing for compressive strength shall be conducted at the same frequency as the acceptance tests for temperature, consistency, and air content.

6-02.3(5)I Vacant

6-02.3(5)J Vacant

6-02.3(5)K Rejecting Concrete

Rejection Without Testing — The Engineer, prior to sampling, may reject any batch or load of concrete that appears defective in composition; such as cement content or aggregate proportions. Rejected material shall not be incorporated in the structure.

6-02.3(5)L Concrete With Non-Conforming Strength

Concrete with cylinder compressive strengths (fc) that fail to meet acceptance level requirements shall be evaluated for structural adequacy. If the material is found to be adequate, payment shall be adjusted in accordance with the following formula:

$$\text{Pay adjustment} = \frac{2(f'c - fc)(UP)(Q)}{f'c}$$

where f′c = Specified minimum compressive strength at 28 days.
fc = Compressive strength at 28 days as determined by AASHTO Test Methods.
UP = Unit contract price per cubic yard for the class of concrete involved.
Q = Quantity of concrete represented by an acceptance test based on the required frequency of testing.

Concrete that fails to meet minimum acceptance levels using the coring method will be evaluated for structural adequacy. If the material is found to be adequate, payment shall be adjusted in accordance with the following formula:

$$\text{Pay adjustment} = \frac{3.56(.85f'c - f\ cores)(UP)(Q)}{f'c}$$

where f’c = Specified minimum compressive strength at 28 days.
f cores = Compressive strength of the cores as determined by AASHTO T-22.
UP = Unit contract price per cubic yard for the class of concrete involved.
Q = Quantity of concrete represented by an acceptance test based on the required frequency of testing.

Where these Specifications designate payment for the concrete on other than a per cubic yard basis, the unit contract price of concrete shall be taken as $300 per cubic yard for concrete Class 4000, 5000, and 6000. For concrete Class 3000, the unit contract price for concrete shall be $160 per cubic yard.

6-02.3(6) Placing Concrete

The Contractor shall not place concrete:

1. On frozen or ice-coated ground or subgrade;
2. Against or on ice-coated forms, reinforcing steel, structural steel, conduits, precast members, or construction joints;
3. Under rainy conditions; placing of concrete shall be stopped before the quantity of surface water is sufficient to affect or damage surface mortar quality or cause a flow or wash the concrete surface;
4. In any foundation until the Engineer has approved its depth and character;
5. In any form until the Engineer has approved it and the placement of any reinforcing in it; or
6. In any work area when vibrations from nearby work may harm the concrete’s initial set or strength.

When a foundation excavation contains water, the Contractor shall pump it dry before placing concrete. If this is impossible, an underwater concrete seal shall be placed that complies with Section 6-02.3(6)B. This seal shall be thick enough to resist any uplift.

All foundations and forms shall be moistened with water just before the concrete is placed. Any standing water on the foundation or in the form shall be removed.

The Contractor shall place concrete in the forms as soon as possible after mixing. The concrete shall always be plastic and workable. For this reason, the Engineer may reduce the time to discharge even further. Concrete placement shall be continuous, with no interruption longer than 30 minutes between adjoining layers unless the Engineer approves a longer time. Each layer shall be placed and consolidated before the preceding layer takes initial set. After initial set, the forms shall not be jarred, and projecting ends of reinforcing bars shall not be disturbed.

In girders or walls, concrete shall be placed in continuous, horizontal layers 1.5 to 2.5-feet deep. Compaction shall leave no line of separation between layers. In each part of a form, the concrete shall be deposited as near its final position as possible.

Any method for placing and consolidating shall not segregate aggregates or displace reinforcing steel. Any method shall leave a compact, dense, and impervious concrete with smooth faces on exposed surfaces. Plastering is not permitted. Any section of defective concrete shall be removed at the Contractor's expense.

To prevent aggregates from separating, the length of any conveyor belt used to transport concrete shall not exceed 300-feet. If the mix needs protection from sun or rain, the Contractor shall cover the belt. When concrete pumps are used for placement, a Contractor's representative shall, prior to use on the first placement of each day, visually inspect the pumps water chamber for water leakage. No pump shall be used that allows free water to flow past the piston.

If a concrete pump is used as the placing system, the pump priming slurry shall be discarded before placement. Initial acceptance testing may be delayed until the pump priming slurry has been eliminated from the concrete being pumped. Eliminating the priming slurry from the concrete may require that several cubic yards of concrete are discharged through the pumping system and discarded. Use of a concrete pump requires a reserve pump (or other backup equipment) at the site.

If the concrete will drop more than 5-feet, it shall be deposited through a sheet metal (or other approved) conduit. If the form slopes, the concrete shall be lowered through approved conduit to keep it from sliding down one side of the form. No aluminum conduits or tremies shall be used to pump or place concrete.

Before placing concrete for roadway slabs on steel spans, the Contractor shall release the falsework under the bridge and let the span swing free on its supports. Concrete in flat slab bridges shall be placed in one continuous operation for each span or series of continuous spans.

Concrete for roadway slabs and the stems of T-beams or box-girders shall be placed in separate operations if the stem of the beam or girder is more than 3-feet deep. First the beam or girder stem shall be filled to the bottom of the slab fillets. Roadway slab concrete shall not be placed until enough time has passed to permit the earlier concrete to shrink (at least 12 hours). If stem depth is 3-feet or less, the Contractor may place concrete in one continuous operation if the Engineer approves.

Between expansion or construction joints, concrete in beams, girders, roadway slabs, piers, columns, walls, and traffic and pedestrian barriers, etc., shall be placed in a continuous operation.

No traffic or pedestrian barrier shall be placed until after the roadway slabs are complete for the entire structure. No concrete barriers shall be placed until the falsework has been released and the span supports itself. The Contractor may choose not to release the deck overhang falsework prior to the barrier placement. The Contractor shall submit calculations to the Engineer indicating the loads induced into the girder webs due to the barrier weight and any live load placed on the structure do not exceed the design capacity of the girder component. This analysis is not required for bridges with concrete superstructures. No barrier, curb, or sidewalk shall be placed on steel or prestressed concrete girder bridges until the roadway slab reaches a compressive strength of at least 3,000 psi.

The Contractor may construct traffic and pedestrian barriers by the slipform method. However, the barrier may not deviate more than $^{1}/_{4}$-inch when measured by a 10-foot straightedge held longitudinally on the front face, back face, and top surface. Electrical conduit within the barrier shall be constructed in accordance with the requirements of Section 8-20.3(5).

When placing concrete in arch rings, the Contractor shall ensure that the load on the falsework remains symmetrical and uniform.

Unless the Engineer approves otherwise, arch ribs in open spandrel arches shall be placed in sections. Small key sections between large sections shall be filled after the large sections have shrunk.

6-02.3(6)A Weather and Temperature Limits to Protect Concrete

Hot Weather Protection

The Contractor shall provide concrete within the specified temperature limits by:

1. Shading or cooling aggregate piles (sprinkling of fine aggregate piles with water is not allowed). If sprinkling of the coarse aggregates is to be used, the piles moisture content shall be monitored and the mixing water adjusted for the free water in the aggregate. In addition, when removing the coarse aggregate, it shall be removed from at least 1 foot above the bottom of the pile.
2. Refrigerating mixing water; or replacing all or part of the mixing water with crushed ice, provided the ice is completely melted by placing time.

If the concrete would probably exceed 90ºF using normal methods, the Engineer may require approved temperature-reduction measures be taken before the placement begins.

If air temperature exceeds 90ºF, the Contractor shall use water spray or other approved methods to cool all concrete-contact surfaces to less than 90ºF. These surfaces include forms, reinforcing steel, steel beam flanges, and any others that touch the mix. The Contractor shall reduce the time between mixing and placing to a minimum and shall not permit mixer trucks to remain in the sun while waiting to discharge concrete. Chutes, conveyors, and pump lines shall be shaded.

If bridge roadway slabs are placed while air temperature exceeds 90ºF, the Contractor shall:

1. Cover the top layer of reinforcing steel with clean, wet burlap immediately before concrete placement;
2. Sprinkle cool water on the forms and reinforcing steel just before the placement if the Engineer requires it;
3. Finish the concrete slab without delay; and
4. Provide at the site water-fogging equipment to be used if needed after finishing to prevent plastic cracks.

If the evaporation rate at the concreting site is 0.20 pounds per square foot of surface per hour or more (determined from Table 6-02.3(6)), the Contractor shall surround the fresh concrete with an enclosure. This enclosure will protect the concrete from wind blowing across its surface until the curing compound is applied. If casting deck concrete that is 80ºF or hotter, the Contractor shall install approved equipment at the site to show relative humidity and wind velocity.

Cold Weather Protection

The Contractor is solely responsible for protecting concrete from inclement weather during the entire curing period. The Contractor shall provide a written procedure for cold weather concreting to the Engineer for review and approval. The procedure shall detail how the Contractor will prevent the concrete temperature from falling below 50º F. Extra protection shall be provided for areas especially vulnerable to freezing (such as exposed top surfaces, corners and edges, thin sections, and concrete placed into steel forms). Permission given by the Engineer to place concrete during cold weather will in no way ensure acceptance of the work by the Contracting Agency. Should the concrete placed under such conditions prove unsatisfactory in any way, the Engineer shall still have the right to reject the work although the plan and the work were carried out with the Engineer's permission.

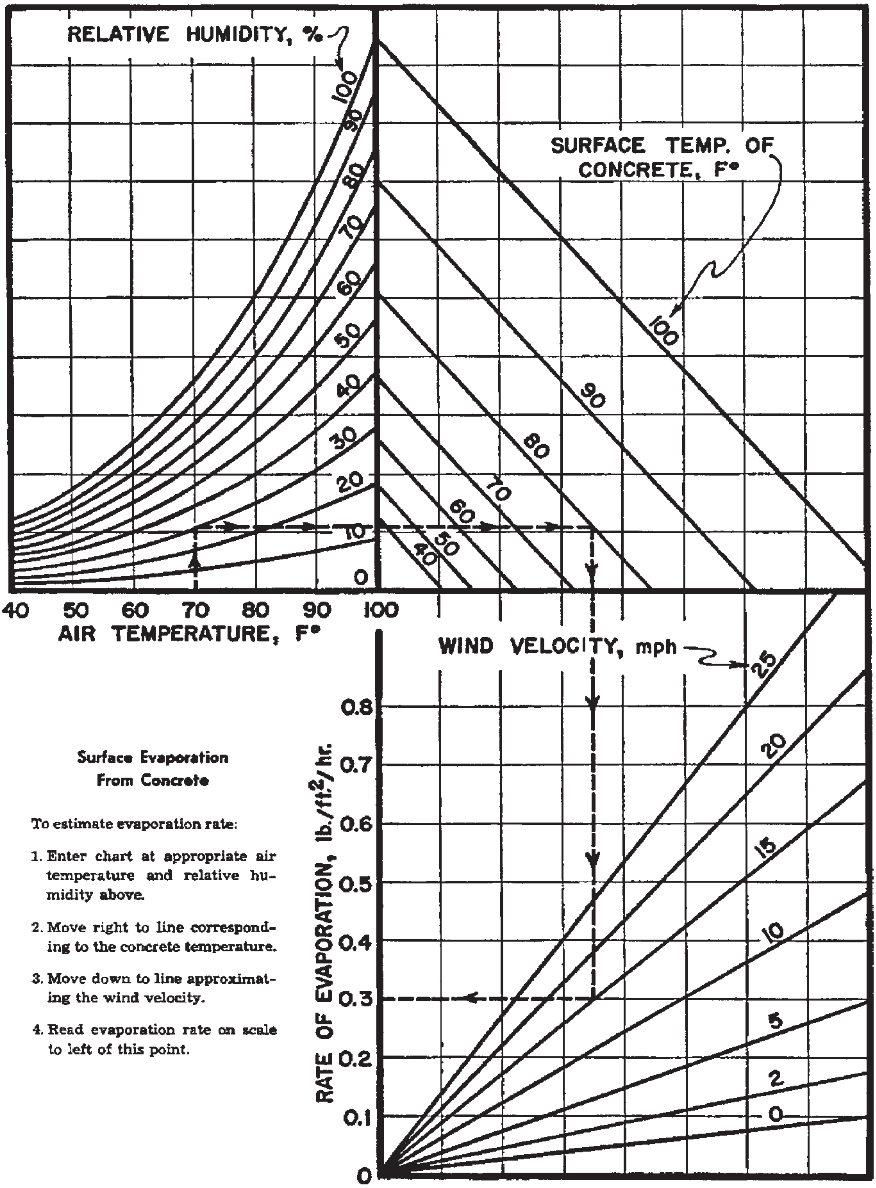
If weather forecasts predict air temperatures below 35º F during the seven days just after the concrete placement, the Contractor may place the concrete only if his approved cold weather concreting plan is implemented.

The Contractor shall provide and maintain a maturity meter in the concrete at a location specified by the Engineer for each concrete placement. During curing, data from the maturity meter shall be readily available to the Engineer. The Contractor shall record and provide time and temperature data on hourly intervals.

The Contractor shall not mix nor place concrete while the air temperature is below 35º F, unless the water or aggregates (or both) are heated to at least 70º F. The aggregate shall not exceed 150º F. If the water is heated to more than 150º F, it shall be mixed with the aggregates before the cement is added. Any equipment and methods shall heat the materials evenly. Concrete placed in shafts and piles is exempt from such preheating requirements.

The Contractor may warm stockpiled aggregates with dry heat or steam, but not by applying flame directly or under sheet metal. If the aggregates are in bins, steam or water coils or other heating methods may be used if aggregate quality is not affected. Live steam heating is not permitted on or through aggregates in bins. If using dry heat, the Contractor shall increase mixing time enough to permit the super-dry aggregates to absorb moisture.

Surface Evaporation from Concrete
Table 6-02.3(6)



Any concrete placed in air temperatures below 35º F shall be immediately protected. In addition to the monitoring of the concrete temperature with a maturity meter the Contractor shall provide recording thermometers or other approved devices to monitor the surface temperature of the concrete. The concrete surface temperature shall be maintained at or above 50º F and the relative humidity shall be maintained above 80 percent. These conditions shall be maintained for a minimum of seven days or for the cure period required by Section 6-02.3(11), whichever is longer. If artificial heat is used to maintain the temperature inside an enclosure, moisture shall be added to the enclosure to maintain the humidity as stated above. The Contractor shall stop adding moisture 24 hours before removing the heat.

If at any period during curing the concrete temperature falls below 50º F on the maturity meter or recording thermometer, no curing time is awarded for that day and the required curing time will be extended day for day where the temperature falls below 50º F. Should the Contractor fail to adequately protect the concrete and the temperature of the concrete falls below 35º F during curing, the Engineer may reject it.

6-02.3(6)B Placing Concrete in Foundation Seals

If the Plans require a concrete seal, the Contractor shall place the concrete underwater inside a watertight cofferdam, tube, or caisson. Seal concrete shall be placed in a compact mass in still water. It shall remain undisturbed and in still water until fully set. While seal concrete is being deposited, the water elevation inside and outside the cofferdam shall remain equal to prevent any flow through the seal in either direction. The cofferdam shall be vented at the vent elevation shown in the Plans. The thickness of the seal is based upon this vent elevation.

The seal shall be at least 18-inches thick unless the Plans show otherwise. The Engineer may change the seal thickness during construction which may require redesign of the footing and the pier shaft or column. Although seal thickness changes may result in the use of more or less concrete, reinforcing steel, and excavation, payment will remain as originally defined in unit contract prices.

To place seal concrete underwater, the Contractor shall use a concrete pump or tremie. The tremie shall have a hopper at the top that empties into a watertight tube at least 10-inches in diameter. The discharge end of the tube on the tremie or concrete pump shall include a device to seal out water while the tube is first filled with concrete. Tube supports shall permit the discharge end to move freely across the entire work area and to drop rapidly to slow or stop the flow. One tremie may be used to concrete an area up to 18-feet per side. Each additional area of this size requires one additional tremie.

Throughout the underwater concrete placement operation, the discharge end of the tube shall remain submerged in the concrete and the tube shall always contain enough concrete to prevent water from entering. The concrete placement shall be continuous until the work is completed, resulting in a seamless, uniform seal. If the concreting operation is interrupted, the Engineer may require the Contractor to prove by core drilling or other tests that the seal contains no voids or horizontal joints. If testing reveals voids or joints, the Contractor shall repair them or replace the seal at no expense to the Contracting Agency.

Concrete Class 4000W shall be used for seals, and it shall meet the consistency requirements of Section 6-02.3(4)C.

6-02.3(6)C Dewatering Concrete Seals and Foundations

After a concrete seal is constructed, the Contractor shall pump the water out of the cofferdam and place the rest of the concrete in the dry. This pumping shall not begin until the seal has set enough to withstand the hydrostatic pressure (three days for gravity seals and ten days for seals containing piling or shafts). The Engineer may extend these waiting periods to ensure structural safety or to meet a condition of the operating permit.

If weighted cribs are used to resist hydrostatic pressure at the bottom of the seal, the Contractor shall anchor them to the foundation seal. Any method used (such as dowels or keys) shall transfer the entire weight of the crib to the seal.

No pumping shall be done during or for 24 hours after concrete placement unless done from a suitable sump separated from the concrete work by a watertight wall. Pumping shall be done in a way that rules out any chance of concrete being carried away.

6-02.3(6)D Protection Against Vibration

Freshly placed concrete shall not be subjected to excessive vibration and shock waves during the curing period until it has reached a 2000 psi minimum compressive strength for concrete Class 4000 and lower strength classes of concrete. For higher strength classes of concrete, the minimum compressive strength for ending the vibration restriction shall be the concrete Class designation (specified in psi) divided by two.

After the first 5 hours from the time the concrete has been placed and consolidated, the Contractor shall keep all vibration producing operations at a safe horizontal distance from the freshly placed concrete by following either the prescriptive safe distance method or the monitoring safe distance method. These requirements for the protection of freshly placed concrete against vibration shall not apply for plant cast concrete, pile driving, shaft installation or soldier pile shaft installation operations, nor shall they apply to the vibrations caused by the traveling public. See Section 6-05.3(11)H, Shaft Special Provisions, and Section 6-16 respectively for pile driving, shaft installation, and soldier pile shaft installation operations.

Prescriptive Safe Distance Method

After the concrete has been placed and consolidated, the Contractor shall keep all vibration producing operations at a safe horizontal distance from the freshly placed concrete as follows:

MINIMUM COMPRESSIVE STRENGTH, f ’c	SAFE HORIZONTAL DISTANCE (1)	
	EQUIPMENT CLASS L (2)	EQUIPMENT CLASS H (3)
< 1000 psi	75-feet	125-feet
1000 psi to < 1400 psi	30-feet	50-feet
1400 psi to 2000 psi	15-feet	25-feet

(1) The safe horizontal distance shall be reduced to 10-feet for small rubber tire construction equipment like backhoes under 50,000 pounds, concrete placing equipment, and legal highway vehicles if such equipment travels at speeds of:

- ≤ 5 mph on relatively smooth roadway surfaces or
- ≤ 3 mph on rough roadway surfaces (i.e. with potholes)

(2) Equipment Class L (Low Vibration) shall include tracked dozers under 85,000 pounds, track vehicles, trucks (unless excluded above), hand operated jack hammers, cranes, auger drill rig, caisson drilling, vibratory roller compactors under 30,000 pounds.

(3) Equipment Class H (High Vibration) shall include machine operated impact tools, pavement breakers, and other large pieces of equipment.

After the concrete has reached a minimum compressive strength specified above, the safe horizontal distance restrictions would no longer apply.

Monitoring Safe Distance Method

The Contractor may monitor the vibration producing operations in order to decrease the safe horizontal distance requirements of the prescriptive safe distance method. If this method is chosen, all construction operations that produce vibration or shock waves in the vicinity of freshly placed concrete shall be monitored by the Contractor with monitoring equipment sensitive enough to detect a minimum peak particle velocity (PPV) of 0.10-inches per second. Monitoring devices shall be placed on or adjacent to the freshly placed concrete when the measurements are taken. During the time subsequent to the concrete placement, the Contractor shall cease all vibration or shock producing operations in the vicinity of the newly placed concrete when the monitoring equipment detects excessive vibration and shock waves defined as exceeding the following PPV's:

MINIMUM COMPRESSIVE STRENGTH, f 'c	MAXIMUM PPV
< 1000 psi	0.10 in/sec
1000 psi to < 1400 psi	1.0 in/sec
1400 psi to 2000 psi	2.0 in/sec

After the concrete has reached a minimum compressive strength specified above, the safe horizontal distance restrictions would no longer apply.

6-02.3(7) Concrete Exposed to Sea Water

If sea water will contact a completed concrete structure, the Contractor shall:

1. Mix the concrete for at least 2 minutes.
2. Control water content to produce concrete that will be as impermeable as possible.
3. Compact the concrete as the Engineer may require, avoiding the formation of any stone pockets.
4. Place only clean, rust-free reinforcement bars in the concrete.
5. Coat form surfaces heavily with shellac and any approved form release agent.
6. Leave forms intact for at least 30 days after concrete placement (longer if the Engineer requires) to prevent sea water from contacting the concrete.
7. Leave the surface of concrete just as it comes from the forms.
8. Provide special handling for any concrete piles used in sea water to avoid even slight deformation cracks.

The Engineer shall decide the range of disintegration possible by exposure to sea water. This range shall extend from a point below the level of extreme low tide up to a point above the level of extreme high tide. Wave action and other conditions will also affect the Engineer's decision on this range. Unless the Engineer approves otherwise, the Contractor shall not locate construction joints within this range. All concrete within this range shall be poured in the dry.

6-02.3(8) Concrete Exposed to Alkaline Soils or Water

The requirements for concrete in seawater shall also apply to concrete in alkaline soils or water. In addition, the Contractor shall:

1. Let the concrete set at least 30 days (longer if possible) before allowing soil or water to contact it directly;
2. Vibrate each batch of concrete immediately after it has been placed into the forms, using enough vibrating tampers to do this effectively; and
3. Hand tamp, if necessary, to produce smooth, dense outside surfaces.

6-02.3(9) Vibration of Concrete

The Contractor shall supply enough vibrators to consolidate the concrete (except that placed underwater) according to the requirements of this section. Each vibrator must:

1. Be designed to operate while submerged in the concrete,
2. Vibrate at a rate of at least 7,000 pulses per minute, and
3. Receive the Engineer's approval on its type and method of use.

Immediately after concrete is placed, vibration shall be applied in the fresh batch at the point of deposit. In doing so, the Contractor shall:

1. Space the vibrators evenly, no farther apart than twice the radius of the visible effects of the vibration;
2. Ensure that vibration intensity is great enough to visibly affect a weight of 1-inch slump concrete across a radius of at least 18-inches;
3. Insert the vibrators slowly to a depth that will effectively vibrate the full depth of each layer, penetrating into the previous layer on multilayer pours;
4. Protect partially hardened concrete (i.e., nonplastic, which prevents vibrator penetration when only its own weight is applied) by preventing the vibrator from penetrating it or making direct contact with steel that extends into it;
5. Not allow vibration to continue in one place long enough to form pools of grout;
6. Continue vibration long enough to consolidate the concrete thoroughly, but not so long as to segregate it;
7. Withdraw the vibrators slowly when the process is complete; and
8. Not use vibrators to move concrete from one point to another in the forms.

When vibrating and finishing top surfaces that will be exposed to weather or wear, the Contractor shall not draw water or laitance to the surface. In high lifts, the top layer shall be shallow and made up of a concrete mix as stiff as can be effectively vibrated and finished.

To produce a smooth, dense finish on outside surfaces, the Contractor shall hand tamp the concrete.

6-02.3(10) Roadway Slabs

A pre-concreting conference shall be held five to ten working days before placing concrete to discuss construction procedures, personnel, and equipment to be used. Those attending shall include:

1. (representing the Contractor) The superintendent and all foremen in charge of placing steel reinforcing bars, of placing the concrete, and of finishing it; and
2. (representing the State) The Project Engineer and key inspection assistants.

If the project includes more than one slab, and if the Contractor's key personnel change between concreting operations, an additional conference shall be held just before each slab is placed.

The Contractor shall not place roadway slabs until the Engineer agrees that:

1. Concrete producing and placement rates will be high enough to meet placing and finishing deadlines,
2. Finishers with enough experience have been employed, and
3. Adequate finishing tools and equipment are at the site.

The finishing machine shall be self-propelled and be capable of forward and reverse movement under positive control. The finishing machine shall be equipped with a rotating cylindrical single or double drum screed not exceeding 60-inches in length. The finishing machine shall have the necessary adjustments to produce the required cross-section, line, and grade. Provisions shall be made for the raising and lowering of all screeds under positive control. The upper vertical limit of screed travel shall permit the screed to clear the finished concrete surface. When placing concrete abutting a previously placed slab, the side of the finishing machine adjacent to the existing slab shall be equipped to travel on the existing slab.

For bridge deck widenings of 20' or less, or where jobsite conditions do not allow the use of conventional configuration finishing machines described above, the Contractor may propose the use of a hand operated motorized power screed such as a "texas" or "bunyon" screed. This screed shall be capable of finishing the deck to the same standards as the finishing machine. The Contractor shall not begin placing deck concrete until receiving the Engineer's approval of this screed and the placing procedures.

The Contractor may use hand-operated strike-boards only when the Engineer approves for special conditions and small areas (less than 10-feet in width and 200-feet in length). These boards must be sturdy and able to strike off the full placement width without intermediate supports. Strike-boards, screed rails, and any specially made auxiliary equipment shall receive the Engineer's approval before use. All finishing requirements in these specifications apply to hand-operated finishing equipment.

Screed rails shall rest on adjustable supports that can be removed with the least possible disturbance to the screeded concrete. The supports shall rest on structural members or on forms rigid enough to resist deflection. Supports shall be removable to at least 2-inches below the finished surface. If possible, the Contractor shall place screed rails outside the finishing area. But if they are placed inside the area, they shall be placed above the finished surface.

Screed rails (with their supports) shall be strong enough and stiff enough to permit the finishing machine to operate effectively on them. All screed rails shall be placed and secured for the full length of the slab before the concreting begins. If the Engineer approves in advance, the Contractor may move rails ahead onto previously set supports while concreting progresses. But such movable rails and their supports shall not change the set elevation of the screed.

On steel truss and girder spans, screed rails and bulkheads may be placed directly on transverse steel floorbeams, with the strike-board moving at right angles to the centerline of the roadway.

Before any concrete is placed, the finishing machine shall be operated over the entire length of the slab to check screed deflection. Concrete placement may begin only if the Engineer approves after this test.

Immediately before placing concrete, the Contractor shall check (and adjust if necessary) all falsework and wedges to minimize settlement and deflection from the added mass of the concrete slab. The Contractor shall also install devices, such as telltales, by which the Engineer can readily measure settlement and deflection.

The Contractor shall schedule the concrete placement so that it can be completely finished during daylight. After dark finishing is permitted if the Engineer approves and if the Contractor provides adequate lighting.

The placement operation shall cover the full width of the roadway or the full width between construction joints. The Contractor shall locate any construction joint over a beam or web that can support the slab on either side of the joint. The joint shall not occur over a pier unless the Plans permit. Each joint shall be formed vertically and in true alignment. The Contractor shall not release falsework or wedges supporting pours on either side of a joint until each side has aged as these Specifications require.

Placement of concrete for slabs shall comply with Section 6-02.3(6). The Engineer shall approve the placement method. In placing the concrete, the Contractor shall:

1. Place it (without segregation) against concrete placed earlier, as near as possible to its final position, approximately to grade, and in shallow, closely spaced piles;
2. Consolidate it around reinforcing steel by using vibrators before strike-off by the finishing machine;
3. Not use vibrators to move concrete;
4. Not revibrate any concrete surface areas where workers have stopped prior to screeding;
5. Remove any concrete splashed onto reinforcing steel in adjacent segments before concreting them;
6. Tamp and strike off the concrete with a template or strike board moving slowly forward at an even speed;
7. Maintain a slight excess of concrete in front of the cutting edge across the entire width of the placement operation;
8. Make enough passes with the strike-board (without bringing excessive amounts of mortar to the surface) to create a surface that is true and ready for final finish; and
9. Leave a thin, even film of mortar on the concrete surface after the last pass of the strike-board.

Workers shall complete all post screeding operations without walking on the concrete. This may require work bridges spanning the full width of the slab.

After removing the screed supports, the Contractor shall fill the voids with concrete (not mortar).

If necessary, as determined by the Engineer, the Contractor shall float the surface left by the finishing machine to remove roughness, minor irregularities, and seal the surface of the concrete. Floating shall leave a smooth and even surface. The floats shall be at least 4-feet long. Each transverse pass of the float shall overlap the previous pass by at least half the length of the float. The first floating shall be at right angles to the strike-off. The second floating shall be at right angles to the centerline of the span. A smooth riding surface shall be maintained across construction joints.

Expansion joints shall be finished with a $^1/_2$-inch radius edger.

After floating, but while the concrete remains plastic, the Contractor shall test the entire slab for flatness (allowing for crown, camber, and vertical curvature). The testing shall be done with a 10-foot straightedge held on the surface. The straightedge shall be advanced in successive positions parallel to the centerline, moving not more than one half the length of the straightedge each time it advances. This procedure shall be repeated with the straightedge held perpendicular to the centerline. An acceptable surface shall be one free from deviations of more than $^1/_8$-inch under the 10-foot straightedge.

If the test reveals depressions, the Contractor shall fill them with freshly mixed concrete, strike off, consolidate, and refinish them. High areas shall be cut down and refinished. Retesting and refinishing shall continue until an acceptable, deviation free surface is produced. The hardened concrete shall meet all smoothness requirements of these Specifications even though the tests require corrective work.

The Contractor will texture the bridge deck by combing the final surface perpendicular to the centerline. Made of a single row of metal tines, the comb shall leave striations in the fresh concrete approximately $^3/_{16}$-inch deep by $^1/_8$-inch wide and spaced approximately $^1/_2$-inch apart. The Engineer will decide actual depths at the site. (If the comb has not been approved, the Contractor shall obtain the Engineer's approval by demonstrating it on a test section.)

The Contractor may operate the combs manually or mechanically, either singly or with several placed end to end. The timing and method used shall produce the required texture without displacing larger particles of aggregate. Texturing shall end 2-feet from curb lines. This 2-foot untextured strip shall be hand finished with a steel trowel.

If the Plans call for an overlay (to be constructed under the same contract), such as hot mix asphalt, latex modified concrete, epoxy concrete, or similar, the Contractor shall produce the final finish by dragging a strip of damp, seamless burlap lengthwise over the full width of the slab or by brooming it lightly. A burlap drag shall equal the slab in width. Approximately 3-feet of the drag shall contact the surface, with the least possible bow in its leading edge. It must be kept wet and free of hardened lumps of concrete. When it fails to produce the required finish, the Contractor shall replace it. When not in use, it shall be lifted clear of the slab.

After the slab has cured, the surface shall not vary more than $^1/_8$-inch under a 10-foot straightedge placed parallel and perpendicular to the centerline.

The Contractor shall cut high spots down with a diamond faced, saw-type cutting machine. This machine shall cut through mortar and aggregate without breaking or dislodging the aggregate or causing spalls.

Low spots shall be built up utilizing a grout or concrete with a strength equal to or greater than the required 28-day strength of the roadway slab. The method of build-up shall be submitted to the Engineer for approval.

The surface texture on any area cut down or built up shall match closely that of the surrounding deck. The entire bridge roadway slab must provide a smooth riding surface.

Concrete for sidewalk slabs shall be well compacted, struck off with a strike-board, and floated with a wooden float to achieve a surface that does not vary more than $^1/_8$ inch under a 10-foot straightedge. An edging tool shall be used to finish all sidewalk edges and expansion joints. The final surface shall have a granular texture that will not turn slick when wet.

6-02.3(11) Curing Concrete

After placement, concrete surfaces shall be cured as follows:

1. Bridge roadway slabs (except those made of concrete Class 4000D), flat slab bridge superstructures, bridge sidewalks, box culvert tops, roofs of cut and cover tunnels — curing compound covered by white, reflective type sheeting or continuous wet curing. Curing by either method shall be for at least 10 days.
2. Class 4000D concrete (regardless of structure type) — two coats of curing compound and continuous wet cure using heavy quilted blankets or burlap for 14 days.
3. All other concrete surfaces (except traffic barriers and rail bases) — continuous moisture for at least three days. When continuous moisture or wet curing is required, the Contractor shall keep the concrete surfaces wet with water during curing.

The Contractor may provide continuous moisture by watering a covering of heavy quilted blankets, by keeping concrete surfaces wet with water continuously and covering with a white reflective type sheeting, or by wetting the outside surfaces of wood forms.

When curing Class 4000D, two coats of curing compound that complies with Section 9-23.2 shall be applied immediately (not to exceed 15 min.) after tining any portion of the deck. The surface shall be covered with presoaked heavy quilted blankets or burlap as soon as the concrete has set enough to allow covering without damaging the finish. Soaker hoses are required and shall be placed on top of burlap or blankets and shall be charged with water frequently to keep the entire deck covering wet during the course of curing.

For all other concrete requiring curing compound, the Contractor shall apply two coats (that complies with Section 9-23.2) to the fresh concrete. The compound shall be applied immediately after finishing. Application of the second coat shall run at right angles to that of the first. The two coats shall total at least 1 gallon per 150 square feet and shall obscure the original color of the concrete. If any curing compound spills on construction joints or reinforcing steel, the Contractor shall clean it off before the next pour.

If the Plans call for an asphalt overlay, the Contractor shall use the clear curing compound (Type 1D), applying at least 1 gallon per 150 square feet to the concrete slab. Otherwise, the Contractor shall use white pigmented curing compound (Type 2), agitating it thoroughly just before and during application. If other materials are to be bonded to the surface, the Contractor shall remove the curing compound by sandblasting or acceptable high pressure water washing.

The Contractor shall have on the site, back-up spray equipment, enough workers, and a bridge from which they will apply the curing compound. The Engineer may require the Contractor to demonstrate (at least one day before the pour) that the crew and equipment can apply the compound acceptably.

The Contractor shall cover the top surfaces with white, reflective sheeting, leaving it in place for at least ten days. Throughout this period, the sheeting shall be kept in place by taping or weighting the edges where they overlap.

The unit contract prices shall cover all concrete curing costs.

6-02.3(11)A Curing and Finishing Concrete Traffic and Pedestrian Barrier

The Contractor shall supply enough water and workers to cure and finish concrete barrier as required in this section. Unit contract prices shall cover all curing and finishing costs.

Fixed-Form Barrier

The edge chamfers shall be formed by attaching chamfer strips to the barrier forms.

After troweling and edging a barrier (while the forms remain in place), the Contractor shall:

1. Brush the top surface with a fine bristle brush;
2. Cover the top surface with heavy, quilted blankets; and
3. Spray water on the blankets and forms at intervals short enough to keep them thoroughly wet for three days.

After removing the forms, the Contractor shall:

1. Remove all lips and edgings with sharp tools or chisels;
2. Fill all holes with mortar;
3. True up corners of openings;
4. Remove concrete projecting beyond the true surface by stoning or grinding;
5. Cover the barrier with heavy, quilted blankets (not burlap);
6. Keep the blankets continuously wet for at least seven days.

The Contractor may do the finishing work described in steps 1 through 4 above during the second (the seven day) curing period if the entire barrier is kept covered except the immediate work area. Otherwise, no finishing work may be done until at least ten days after pouring.

After the ten day curing period, the Contractor shall remove from the barrier all form-release agent, mud, dust, and other foreign substances in either of two ways: (1) by light sandblasting and washing with water, or (2) by spraying with a high-pressure water jet. The water jet equipment shall use clean fresh water and shall produce (at the nozzle) at least 1,500 psi with a discharge of at least 3 gpm. The water jet nozzle shall have a 25 degree tip and shall be held no more than 9-inches from the surface being washed.

After cleaning, the Contractor shall use brushes to rub 1:1 mortar into air holes and small crevices on all surfaces except the brushed top. This mortar shall consist of one part Portland cement (of the same brand used in the concrete) and one part clean, fine plaster sand. As soon as the mortar takes its initial set, the Contractor shall rub it off with a piece of sacking or carpet. The barrier shall then be covered with wet blankets for at least 48 hours.

No curing compound shall be used on fixed-form concrete barrier. The completed surface of the concrete shall be even in color and texture.

Slip-Form Barrier

The edge radius shall be formed by attaching radius strips to the barrier slip form.

The Contractor shall finish slip-form barrier by: (1) steel troweling to close all surface pockmarks and holes; and (2) for plain surface barrier, lightly brushing the front and back face with vertical strokes and the top surface with transverse strokes.

After finishing, the Contractor shall cure the slip-form barrier by using either method A (curing compound) or B (wet blankets) described below.

Method A. Under the curing compound method, the Contractor shall:

1. Spray two coats of clear curing compound (Type 1D) on the concrete surface after the free water has disappeared. (Coverage of combined coats shall equal at least 1 gallon per 150 square feet.)
2. No later than the morning after applying the curing compound, cover the barrier with white, reflective sheeting for at least ten days.
3. After the ten day curing period, remove the curing compound as necessary by light sandblasting or by spraying with a high-pressure water jet to produce an even surface appearance. The water jet equipment shall use clean fresh water and shall produce (at the nozzle) at least 2,500 psi with a discharge of at least 4 gpm. The water jet nozzle shall have a 25-degree tip and shall be held no more than 9-inches from the surface being cleaned. The Contractor may propose to use a curing compound/concrete sealer. The Engineer will evaluate the proposal and if found acceptable, shall approve the proposal in writing. As a minimum, the Contractor's proposal shall include:

    Product identity

    Manufacturer's recommended application rate

    Method of application and necessary equipment

    Material Safety Data Sheet (MSDS)

    Sample of the material for testing

   Allow 14 working days for evaluating the proposal and testing the material.

Method B. Under the wet cure method, the Contractor shall:

1. Provide an initial cure period by continuous fogging or mist spraying for at least the first 24 hours.
2. After the initial cure period, cover the barrier with a heavy quilted blanket.
3. Keep the blankets continuously wet for at least ten days. (No additional finishing is required at the end of the curing period.)

6-02.3(12) Construction Joints

If the Engineer approves, the Contractor may add, delete, or relocate construction joints shown in the Plans. Any request for such changes shall be in writing, accompanied by a drawing that depicts them. The Contractor will bear any added costs that result from such changes.

All construction joints shall be formed neatly with grade strips or other approved methods. The Contracting Agency will not accept irregular or wavy pour lines. Wire mesh forming material shall not be used. All joints shall be horizontal, vertical, or perpendicular to the main reinforcement. The Contractor shall not use an edger on any construction joint, and shall remove any lip or edging before making the adjacent pour.

If the Plans require a roughened surface on the joint, the Contractor shall strike it off to leave grooves at right angles to the length of the member. The grooves shall be $^{1}/_{2}$-inch to 1-inch wide, $^{1}/_{4}$-inch to $^{1}/_{2}$-inch deep, and spaced equally at twice the width of the groove. If the first strike-off does not produce the required roughness, the Contractor shall repeat the process before the concrete reaches initial set. The final surface shall be clean and without laitance or loose material.

If the Plans do not require a roughened surface, the Contractor shall include shear keys at all construction joints. These keys shall provide a positive, mechanical bond. Shear keys shall be formed depressions and the forms shall not be removed until the concrete has been in place at least 12 hours. Forms shall be slightly beveled to ensure ready removal. Raised shear keys are not allowed.

Shear keys for the tops of beams, at tops and bottoms of boxed girder webs, in diaphragms, and in crossbeams shall:

1. Be formed with 2- by 8-inch wood blocks;
2. Measure 8-inches lengthwise along the beam or girder stem;
3. Measure 4-inches less than the width of the stem, beam, crossbeam, etc. (measured transverse of the stem); and
4. Be spaced at 16-inches center to center.

Unless the Plans show otherwise, in other locations (not named above), shear keys shall equal approximately one third of the joint area and shall be approximately $1^1/_2$-inches deep.

Before placing new concrete against cured concrete, the Contractor shall thoroughly clean and roughen the cured face and wet it with water. Before placing the reinforcing mat for footings on seals, the Contractor shall: (1) remove all scum, laitance, and loose gravel and sediment; (2) clean the construction joint at the top of the seals; and (3) chip off any high spots on the seals that would prevent the footing steel from being placed in the position required by the Plans.

6-02.3(13) Expansion Joints

This section outlines the requirements of specific expansion joints shown in the Plans. The Plans may require other types of joints, seals, or materials than those described here.

Joints made of a vulcanized, elastomeric compound (with neoprene as the only polymer) shall be installed with an approved lubricant adhesive as recommended by the manufacturer. The length of a seal shall match that required in the Plans without splicing or stretching.

Open joints shall be formed with a template made of wood, metal, or other suitable material. Insertion and removal of the template shall be done without chipping or breaking the edges or otherwise injuring the concrete.

Any part of an expansion joint running parallel to the direction of expansion shall provide a clearance of at least $^1/_2$-inch (produced by inserting and removing a spacer strip) between the two surfaces. The Contractor shall ensure that the surfaces are precisely parallel to prevent any wedging from expansion and contraction.

All poured rubber joint sealer (and any required primer) shall conform with Section 9-04.2(2).

6-02.3(14) Finishing Concrete Surfaces

All concrete shall show a smooth, dense, uniform surface after the forms are removed. If it is porous, the Contractor shall bear the cost of repairing it. The Contractor shall clean and refinish any stained or discolored surfaces that may have resulted from their work or from construction delays.

Subsections A and B (below) describe two classes of surface finishing.

6-02.3(14)A Class 1 Surface Finish

The Contractor shall apply a Class 1 finish to all surfaces of concrete members to the limits designated in the contract plans.

The Contractor shall follow steps 1 through 8 below. When steel forms have been used and when the surface of filled holes matches the texture and color of the area around them, the Contractor may omit steps 3 through 8. To create a Class 1 surface, the Contractor shall:

1. Remove all bolts and all lips and edgings where form members have met;
2. Fill all holes greater than $^1/_4$-inch with 1:2 mortar floated to an even, uniform finish;
3. Thoroughly wash the surface of the concrete with water;
4. Brush on a 1:1 mortar, working it well into the small air holes and other crevices in the face of the concrete;
5. Brush on no more mortar than can be finished in one day;
6. Rub the mortar off with burlap or a piece of carpet as soon as it takes initial set (before it reaches final set);
7. Fog-spray water over the finish as soon as the mortar paint has reached final set; and
8. Keep the surface damp for at least two days.

If the mortar becomes too hard to rub off as described in step 6, the Contractor shall remove it with a Carborundum stone and water. Random grinding is not permitted.

6-02.3(14)B Class 2 Surface Finish

The Contractor shall apply a Class 2 finish to all above-ground surfaces not receiving a Class 1 finish as specified above unless otherwise indicated in the contract. Surfaces covered with fill do not require a surface finish.

To produce a Class 2 finish, the Contractor shall remove all bolts and all lips and edgings where form members have met and fill all form tie holes.

6-02.3(15) Date Numerals

Standard date numerals shall be placed where shown in the Plans. The date shall be for the year in which the structure is completed. When traffic barrier is placed on an existing structure, the date shall be for the year in which the original structure was completed. Unit contract prices shall cover all costs relating to these numerals.

6-02.3(16) Plans for Falsework and Formwork

The Contractor shall submit all plans for falsework and formwork for approval or preapproval directly to the Bridge and Structures Office, Construction Support Engineer, Washington State Department of Transportation, PO Box 47340, Olympia, WA 98504-7340 as described in Section 6-02.3(16)A or 6-02.3(16)B. The Contractor shall also submit two sets of the falsework and formwork plans to the Project Engineer. Approval will not reduce the Contractor's responsibility for ensuring the adequacy of the formwork and falsework. All falsework and formwork shall be constructed in accordance with approved falsework and formwork plans.

Except for the placement of falsework foundation pads and piles, the construction of any unit of falsework shall not start until the Engineer has reviewed and approved the falsework plans for that unit. Falsework driven piling, temporary concrete footings, or timber mudsills may be placed as described in Section 6-02.3(17)D prior to approval at the Contractor's own risk, except for the following conditions:

1. The falsework is over or adjacent to roadways or railroads as described in Section 6-02.3(17)C, or
2. The falsework requires prior placement of shoring or cofferdams as described in Section 2-09.3(3)D.

Costs associated with modifying falsework to bring it into compliance with the approved falsework plans shall be at the Contractor's expense.

If the project involves a railroad or the U.S. Bureau of Reclamation, additional sets for the portion of the project that involves them shall be sent to:

US Postal Service:

Washington State Department of Transportation
Bridge and Structures Engineer
Construction Support
PO Box 47340
Olympia WA 98504-7340

FedEx:

Washington State Department of Transportation
Bridge and Structures Engineer
Construction Support
4500 3rd Avenue SE
Lacey WA 98503

1. Four sets for each railroad company affected, and
2. Six sets for the U.S. Bureau of Reclamation.

The Department will review the falsework and formwork plans and calculations, and if they are acceptable, will obtain the required approvals from the appropriate railroad company or the U.S. Bureau of Reclamation. After the Department has received approval and any comments from the railroad company or the U.S. Bureau of Reclamation, two copies of the falsework and formwork plans will then be marked with any comments and returned to the Contractor.

Plan approval is not required for footing or retaining walls unless they are more than 4-feet high (excluding pedestal height).

The design of falsework and formwork shall be based on:

1. Applied loads and conditions which are no less severe than those described in Section 6-02.3(17)A, "Design Loads;"
2. Allowable stresses and deflections which are no greater than those described in Section 6-02.3(17)B, "Allowable Stresses and Deflections;"
3. Special loads and requirements no less severe than those described in Section 6-02.3(17)C, "Falsework and Formwork at Special Locations;" and
4. Conditions required by other Sections of 6-02.3(17), Falsework and Formwork." Plan approval can be done by the Project Engineer for footings and walls 4-feet to 8-feet high (excluding pedestal height) provided:
5. Concrete placement rate is 4-feet per hour or less.
6. Facing is $^{3}/_{4}$-inch plywood with grade as specified per Section 6-02.3(17)J.
7. Studs, with plywood face grain perpendicular, are 2x4's spaced at 12-inches.
8. Walers with 3,000 pound safe working load ties spaced at 24-inches are 2-2x4's spaced at 24-inches.

Plan approval can be done by the Project Engineer for manufactured certified steel round column forming for column heights up to 20-feet. Concrete placement rate shall not exceed 10-feet per hour. Bracing requirements shall be per manufacturer's recommendations or submitted according to Section 6-02.3(16).

The falsework and formwork plans shall be scale drawings showing the details of proposed construction, including: sizes and properties of all members and components; spacing of bents, posts, studs, wales, stringers, wedges and bracing; rates of concrete placement, placement sequence, direction of placement, and location of construction joints; identify falsework devices and safe working load as well as identifying any bolts or threaded rods used with the devices including their diameter, length, type, grade, and required torque. Show in the falsework plans the proximity of falsework to utilities or any nearby structures including underground structures. Formwork accessories shall be identified according to Section 6-02.3(17)H, "Formwork Accessories." All assumptions, dimensions, material properties, and other data used in making the structural analysis shall be noted on the drawing.

The Contractor shall furnish two copies of the associated design calculations to the Bridge and Structures Office, Construction Support Engineer for examination as a condition for approval. The design calculations shall show the stresses and deflections in load supporting members. Construction details which may be shown in the form of sketches on the calculation sheets shall be shown in the falsework or formwork drawings as well. Falsework or formwork plans will not be approved in any case where it is necessary to refer to the calculation sheets for information needed for complete understanding of the falsework and formwork plans or how to construct the falsework and formwork.

All falsework and formwork plans and design calculations submitted to the Bridge and Structures Office shall be prepared by (or under the direct supervision of) a Professional Engineer, licensed under Title 18 RCW, State of Washington, in the branch of Civil or Structural Engineering.

Each sheet of falsework and formwork plans shall carry the following:

1. Professional Engineer's original signature, date of signature, original seal, registration number, and date of expiration.
2. The initials and dates of all participating design professionals.
3. Clear notation of all revisions including identification of who authorized the revision, who made the revision, and the date of the revision.
4. The contract number, contract title, and sequential sheet number. These shall also be on any related documents.
5. Identify where the falsework and formwork plan will be utilized by referencing Contract Plan sheet number and related item or detail.

Design calculations shall carry on the cover page, the Professional Engineer's original signature, date of signature, original seal, registration number, and date of expiration. The cover page shall include the contract number, contract title, and sequential index to calculation page numbers.

A State of Washington Professional Engineer, licensed under Title 18 RCW, State of Washington, in the branch of Civil or Structural Engineering may be retained to check, review and certify falsework and formwork plans and calculations of an individual who is licensed in another state provided that the following conditions are satisfied:

1. That the work being reviewed was legally prepared by an individual holding valid registration in another state as a civil or structural engineer.
2. The Washington State Professional Engineer conducts independent calculations and reviews all technical matters contained within the subject work, falsework and formwork plans, Contract Plans, Specifications, legal requirements, technical standards, other related documents; and has verified that the design meets all applicable specifications and is in agreement with the specific site conditions and geometry.
3. All falsework and formwork plan sheets shall carry the Washington State Professional Engineer's original signature, date of signature, original seal, registration number, and date of expiration.
4. Two copies of the Washington State Professional Engineer's independent calculations shall be submitted to the Bridge and Structures Office, Construction Support Engineer for review along with the falsework and formwork plans. The independent calculations shall carry on the cover page the Washington State Professional Engineer's original signature, date of signature, original seal, registration number, and date of expiration. The cover page shall include the following: the contract number, contract title, and sequential index to calculation page numbers
5. The Washington State Professional Engineer shall keep, a signed and sealed copy of the falsework, formwork plans, independent calculations, specifications and other related documentation that represents the extent of the review.

6-02.3(16)A Nonpreapproved Falsework and Formwork Plans

The Contractor shall submit six copies of all non-preapproved falsework and formwork plans, and two copies of the design calculations, directly to the following for review and approval and submit two copies of the falsework and formwork plans to the Project Engineer.

US Postal Service:

Washington State Department of Transportation
Bridge and Structures Engineer
Construction Support
PO Box 47340
Olympia WA 98504-7340

FedEx:

Washington State Department of Transportation
Bridge and Structures Engineer
Construction Support
4500 3rd Avenue SE
Lacey WA 98503

Reviewed falsework and formwork plans will be returned from the Bridge and Structures Office, Construction Support Engineer to the Project Engineer who will forward them to the Contractor within the time allowed according to Section 6-01.9. The time allowed begins when the Contractor's transmittal and submittal including all required copies of the falsework and/or formwork plans and calculations, catalog data, and other technical information are received by the Bridge and Structures Office, Construction Support Engineer. Fax copies are considered only informational. For multiple submittals or multiple parts to the same submittal and priority of review see Section 6-01.9.

Plans returned to the Contractor for correction shall be corrected and clean (without any previous WSDOT stamps and comments) revised falsework and formwork plans resubmitted to the Bridge and Structures Office, Construction Support Engineer for review and approval.

The Contractor may revise approved falsework and formwork plans, provided sufficient time is allowed for the Engineer's review and approval before construction is started on the revised portions. Such additional time will not be more than that which was originally allowed per Section 6-01.9. After a plan or drawing is approved and returned to the Contractor, all changes that the Contractor proposed shall be submitted to the Project Engineer for review and approval.

6-02.3(16)B Preapproved Formwork Plans

The Contractor may request preapproval on formwork plans for abutments, wingwalls, diaphragms, retaining walls, columns, girders and beams, box culverts, railings, and bulkheads. Plans for falsework supporting the roadway slab for interior spans between precast prestressed concrete girders may also be submitted for preapproval. Other falsework plans, however, will not be preapproved, but shall be submitted for review and approval as required in Section 6-02.3(16)A.

To apply for preapproval, the Contractor shall submit one reproducible drawing for each formwork plan sheet and two copies of the design calculations directly to:

US Postal Service:

Washington State Department of Transportation
Bridge and Structures Engineer
Construction Support
PO Box 47340
Olympia WA 98504-7340

Fedex:

Washington State Department of Transportation
Bridge and Structures Engineer
Construction Support
4500 3rd Avenue SE
Lacey WA 98503

The Bridge and Structures Office, Construction Support Engineer will return the formwork plan to the Contractor stamped "Preapproved" with an effective date of approval or will indicate any changes required for approval. The reviewed formwork plan will be returned from the Bridge and Structures Office, Construction Support Engineer to the Contractor within the time allowed according to Section 6-01.9. The time allowed begins when the Contractor's transmittal and submittal including all required information are received by the Bridge and Structures Office, Construction Support Engineer.

For each contract on which the preapproved formwork plans will be used, the Contractor shall submit three copies to the Project Engineer. Construction shall not begin until the Project Engineer has given approval.

If the forms being constructed have any deviations to the preapproved formwork plan, the Contractor shall submit formwork plan revisions for review and approval per Section 6-02.3(16)A.

6-02.3(17) Falsework and Formwork

Formwork and falsework are both structural systems. Formwork contains the lateral pressure exerted by concrete placed in the forms. Falsework supports the vertical and/or the horizontal loads of the formwork, reinforcing steel, concrete, and live loads during construction.

The Contractor shall set falsework, to produce in the finished structure, the lines and grades indicated in the Contract Plans. The setting of falsework shall allow for shrinkage, settlement, falsework girder camber, and any structural camber the Plans or the Engineer require.

Concrete forms shall be mortar tight, true to the dimensions, lines, and grades of the structure. Curved surfaces shown in the Contract Plans shall be constructed as curved surfaces and not chorded, except as allowed in Section 6-02.3(17)J. Concrete formwork shall be of sufficient strength and stiffness to prevent overstress and excess deflection as defined in Section 6-02.3(17)B. The rate of depositing concrete in the forms shall not exceed the placement rate in the approved formwork plan. The interior form shape and dimensions shall also ensure that the finished concrete will conform with the Contract Plans.

If the new structure is near or part of an existing one, the Contractor shall not use the existing structure to suspend or support falsework unless the Plans or Special Provisions state otherwise. For prestressed girder and T-beam bridge widenings or stage construction, the roadway deck and the diaphragm forms may be supported from the existing structure or previous stage, if approved by the Engineer. For steel plate girder bridge widenings or stage construction, only the roadway deck forms may be supported from the existing structure or previous stage, if approved by the Engineer. See Section 6-02.3(17)E for additional conditions.

On bridge roadway slabs, forms designed to stay in place made of steel or precast concrete panels shall not be used.

For post-tensioned structures, both falsework and forms shall be designed to carry the additional loads caused by the post-tensioning operations. The Contractor shall construct supporting falsework in a way that leaves the superstructure free to contract and lift off the falsework during post-tensioning. Forms that will remain inside box girders to support the placement of the roadway slab concrete shall, by design, resist girder contraction as little as possible. See Section 6-02.3(26) for additional conditions.

6-02.3(17)A Design Loads

The design load for falsework shall consist of the sum of dead and live vertical loads, and a design horizontal load. The minimum total design load for any falsework shall not be less than 100 lbs./sf. for combined live and dead load regardless of structure thickness.

The entire superstructure cross-section, except traffic barrier, shall be considered to be placed at one time for purposes of determining support requirements and designing falsework girders for their stresses and deflections, except as follows:

> For concrete box girder bridges, the girder stems, diaphragms, crossbeams, and connected bottom slabs, if the stem wall is placed more than 5 days prior to the top slab, may be considered to be self supporting between falsework bents at the time the top slab is placed, provided that the distance between falsework bents does not exceed 4 times the depth of the portion of the girder placed in the preceding concrete placements.

Falsework bents shall be designed for the entire live load and dead load, including all load transfer that takes place during post-tensioning, and braced for the design horizontal load.

Dead loads shall include the weight of all successive placements of concrete, reinforcing steel, forms and falsework, and all load transfer that takes place during post-tensioning. The weight of concrete with reinforcing steel shall be assumed to be not less than 160 pounds per cubic foot.

Live loads shall consist of the actual mass of any equipment to be supported by falsework applied as concentrated loads at the points of contact, and a minimum uniform load of not less than 25 lbs./sf. applied over the entire falsework plan area supported, plus a minimum load of not less than 75 pounds per linear foot applied at the outside edge of deck overhangs.

The design horizontal load to be resisted by the falsework bracing system in any direction shall be:

> The sum of all identifiable horizontal loads due to equipment, construction sequence, side-sway caused by geometry or eccentric loading conditions, or other causes, and an allowance for wind plus an additional allowance of 1 percent of the total dead load to provide for unexpected forces. In no case shall the design horizontal load be less than three percent of the total dead load.

The minimum horizontal load to be allowed for wind on each heavy-duty steel shoring tower having a vertical load carrying capacity exceeding 30 kips per leg shall be the sum of the products of the wind impact area, shape factor, and the applicable wind pressure value for each height zone. The wind impact area is the total projected area of all the elements in the tower face normal to the applied wind. The shape factor for heavy-duty steel shoring towers shall be taken as 2.2. Wind pressure values shall be determined from the following table:

Wind Pressure on Heavy-Duty Steel Shoring Towers

Height Zone (Feet Above Ground)	Wind Pressure Value	
	Adjacent to Traffic	At Other Locations
0 to 30	20 psf	15 psf
30 to 50	25 psf	20 psf
50 to 100	30 psf	25 psf
Over 100	35 psf	30 psf

The minimum horizontal load to be allowed for wind on all other types of falsework, including falsework girders and forms supported on heavy-duty steel shoring towers, shall be the sum of the products of the wind impact area and the applicable wind pressure value for each height zone. The wind impact area is the gross projected area of the falsework support system, falsework girders, forms and any unrestrained portion of the permanent structure, excluding the areas between falsework posts or towers where diagonal bracing is not used. Wind pressure values shall be determined from the following table:

Wind Pressure on All Other Types of Falsework		
	Wind Pressure Value	
Height Zone (Feet Above Ground)	For Members Over and Bents Adjacent to Traffic Openings	At Other Locations
0 to 30	2.0 Q psf	1.5 Q psf
30 to 50	2.5 Q psf	2.0 Q psf
50 to 100	3.0 Q psf	2.5 Q psf
Over 100	3.5 Q psf	3.0 Q psf

The value of Q in the above tabulation shall be determined as follows:

$Q = 1 + 0.2W$; but Q shall not be more than 10.

Where:

W is the width of the falsework system, in feet, measured normal to the direction of the wind force being considered.

The falsework system shall also be designed so that it will be sufficiently stable to resist overturning prior to the placement of the concrete. The minimum factor of safety against falsework overturning in all directions from the assumed horizontal load for all stages of construction shall be 1.25. If the required resisting moment is less than 1.25 times the overturning moment, the difference shall be resisted by bracing, cable guys, or other means of external support.

Design of falsework shall include the vertical component (whether positive or negative) of bracing loads imposed by the design horizontal load. Design of falsework shall investigate the effects of any horizontal displacement due to stretch of the bracing. This is particularly important when using cable or rod bracing systems.

If the concrete is to be post-tensioned, the falsework shall be designed to support any increased or redistributed loads caused by the prestressing forces.

6-02.3(17)B Allowable Design Stresses and Deflections

The maximum allowable stresses listed in this Section are based on the use of identifiable, undamaged, high-quality materials. Stresses shall be appropriately reduced if lesser quality materials are to be used.

These maximum allowable stresses include all adjustment factors, such as the short term load duration factor. The maximum allowable stresses and deflections used in the design of the falsework and formwork shall be as follows:

Deflection

Deflection resulting from dead load and concrete pressure for exposed visible surfaces, $^1/_{360}$ of the span.

Deflection resulting from dead load and concrete pressure for unexposed non-visible surfaces, including the bottom of the deck slab between girders, $^1/_{270}$ of the span.

In the foregoing, the span length shall be the center line to center line distance between supports for simple and continuous spans, and from the center line of support to the end of the member for cantilever spans. For plywood supported on members wider than $1^1/_2$-inches, the span length shall be taken as the clear span plus $1^1/_2$-inches. Also, dead load shall include the weight of all successive placements of concrete, reinforcing steel, forms and falsework self weight. Only the self weight of falsework girders may be excluded from the calculation of the above deflections provided that the falsework girder deflection is compensated for by the installation of camber strips.

Where successive placements of concrete are to act compositely in the completed structure, deflection control becomes extremely critical. Maximum deflection of supporting members — $^1/_{500}$ of the span for members constructed in several successive placements (such as concrete box girder and concrete T-beam girder structures) falsework components shall be sized, positioned, and/or supported to minimize progressive increases in deflection of the structure which would preload the concrete or reinforcing steel before it becomes fully composite.

Timber

Each species and grade of timber/lumber used in constructing falsework and formwork shall be identified in the drawings. The allowable stresses and loads shall not exceed the lesser of stresses and loads given in the table below or factored stresses for designated species and grade in Table 7.3 of the *Timber Construction Manual, Third Edition* by the American Institute of Timber Construction.

Compression perpendicular to the grain reduced to 300 psi for use when moisture content is 19 percent or more (areas exposed to rain, concrete curing water, green lumber).	450 psi
Compression parallel to the grain but not to exceed 1,500 psi.	$\frac{480{,}000 \text{ psi}}{(L/d)^2}$
Flexural stress for members with a nominal depth greater than 8-inches.	1,800 psi
Flexural stress psi for members with a nominal depth of 8-inches or less.	1,500 psi
The maximum horizontal shear.	140 psi
AXIAL tension.	1,200 psi
The maximum modulus of elasticity (E) for timber.	1,600,000 psi

Where:

L is the unsupported length; and

d is the least dimension of a square or rectangular column, or the width of a square of equivalent cross-sectional area for round columns.

The allowable stress for compression perpendicular to the grain, and for horizontal shear shall not be increased by any factors such as short duration loading. Additional requirements are found in other parts of Section 6-02.3(17). Criteria for the design of lumber and timber connections are found in Section 6-02.3(17)I.

Plywood for formwork shall be designed in accordance with the methods and stresses allowed in the *APA Design/Construction Guide for Concrete Forming* as published by the American Plywood Association, Tacoma, Washington. As concrete forming is a special application for plywood, wet stresses shall be used and then adjusted for forming conditions such as duration of load, and experience factors. Concrete pour pressures shall be per Section 6-02.3(17)J.

Steel

For identified grades of steel, design stresses shall not exceed those specified in the *Manual of Steel Construction - Allowable Stress Design, Ninth Edition* by the American Institute of Steel Construction, except as follows:

Compression, flexural but not to exceed $0.6F_y$	$\frac{12{,}000{,}000 \text{ psi}}{Ld/bt}$
The modulus of elasticity (E) shall be	29,000,000 psi

When the grade of steel cannot be positively identified as with salvaged steel and if rivets are present, design stresses shall not exceed the following:

Yield point f_y	30,000 psi
Tension, axial, and flexural	16,000 psi
Compression, axial except L/r shall not exceed 120	$14{,}150 - 0.37(KL/r)^2$ psi
Shear on gross section of the web of rolled shapes	9,500 psi
Web crippling for rolled shapes	22,500 psi
Compression, flexural but not to exceed 16,000 psi and L/b not greater than 39	$16{,}000 - 5.2(L/b)^2$ psi
The modulus of elasticity (E) shall be	29,000,000 psi

Where:

- L is the unsupported length;
- d is the least dimension of rectangular columns, or the width of a square of equivalent cross-sectional area for round columns, or the depth of beams;
- b is the flange width;
- t is the thickness of the compression flange;
- r is the radius of gyration of the compression flange about the weak axis of the member; and
- F_y is the specified minimum yield stress, psi, for the grade of steel used.

All dimensions are expressed in inches.

6-02.3(17)C Falsework and Formwork at Special Locations

In addition to the minimum requirements specified in Sections 6-02.3(17)A and 6-02.3(17)B, falsework towers or posts supporting beams directly over roadways or railroads which are open to traffic or the public shall be designed and constructed so that the falsework will be stable if subjected to impact by vehicles. The use of damaged materials, unidentifiable material, salvaged steel or steel with burned holes or questionable weldments shall not be used for falsework described in this section. For the purposes of this specification the following public or private facilities shall also be considered as “roadways”: pedestrian pathways and other structures such as bridges, walls, and buildings.

The dimensions of the clear openings to be provided through the falsework for roadways, railroads, or pedestrian pathways shall be as specified in the Contract.

Falsework posts or shoring tower systems which support members that cross over a roadway or railroad shall be considered as adjacent to roadways or railroads. Other falsework posts or shoring towers shall be considered as adjacent to roadways or railroads only if the following conditions apply:

1. Located in the row of falsework posts or shoring towers nearest to the roadway or railroad; and
2. Horizontal distance from the traffic side of the falsework to the edge of pavement is less than the total height of the falsework and forms; or
3. The total height of the falsework and forms is greater than the horizontal clear distance between the base of the falsework and a point 10-feet from the centerline of track.

The Contractor shall provide any additional features for the work needed to ensure that the falsework will be stable for impact by vehicles; providing adequate safeguards, safety devices, protective equipment, and any other needed actions to protect property and the life, health, and safety of the public; and shall comply with the provisions in Section 1-07.23 and Section 6-02.3(17)M. The falsework design at special locations, shall incorporate the minimum requirements detailed in this Section, even if protected by concrete median barrier.

The vertical load used for the design of falsework posts and towers which support the portion of the falsework over openings, shall be the greater of the following:

1. 150 percent of the design load calculated in accordance with Section 6-02.3(17)B, but not including any increased or redistributed loads caused by the post-tensioning forces; or
2. 100 percent of the design load plus the increased or redistributed loads caused by the post-tensioning forces.

Each falsework post or each shoring tower leg adjacent to roadways or railroads shall consist of either steel with a minimum section modulus about each axis of 9.5-inches cubed or sound timbers with a minimum section modulus about each axis of 250-inches cubed.

Each falsework post or shoring tower leg adjacent to roadways or railroads shall be mechanically connected to its supporting footing at its base, or otherwise laterally restrained, to withstand a force of not less than 2,000 pounds applied at the base of the post or tower leg in any direction except toward the roadway or railroad track. Posts or tower legs shall be connected to the falsework cap and stringer by mechanical connections capable of resisting a load in any horizontal direction of not less than 1,000 pounds.

For falsework spans over roadways and railroads, all falsework stringers shall be mechanically connected to the falsework cap or framing. The mechanical connections shall be capable of resisting a load in any direction, including uplift on the stringer, of not less than 500 pounds. All associated connections shall be installed before traffic is allowed to pass beneath the span.

When timber members are used to brace falsework bents which are located adjacent to roadways or railroads, all connections shall be bolted through the members using $^{5}/_{8}$-inch diameter or larger bolts.

Concrete traffic barrier shall be used to protect all falsework adjacent to traveled roadways. The falsework shall be located so that falsework footings, mudsills, or piles are at least 2-feet clear of the traffic barrier and all other falsework members shall also be at least 2-feet clear of the traffic barrier. Traffic barrier used to protect falsework shall not be fastened, guyed, or blocked to any falsework but shall be fastened to the pavement according to details shown in the Plans. The installation of concrete traffic barrier shall be completed before falsework erection is begun. The traffic barrier at the falsework shall not be removed until approved by the Engineer. Falsework openings which are provided for the Contractor's own use (not for public use) shall also use concrete traffic barrier to protect the falsework, except the minimum clear distance between the barrier and falsework footings, mudsills, piles, or other falsework members shall be at least 3-inches.

Falsework bents within 20-feet of the center line of a railroad track shall be braced to resist the required horizontal load or 2,000 pounds whichever is greater.

Pedestrian openings through falsework shall be paved or surfaced with full width continuous wood walks which shall be wheel chair accessible and shall be kept clear. Pedestrians shall be protected from falling objects and water falling from construction above. Overhead protection for pedestrians shall extend at least 4-feet beyond the edge of the bridge deck. Plans and details of the overhead protection and pathway shall be submitted with the falsework plans for review and approval. Pedestrian openings through falsework shall be illuminated by temporary lighting, constructed and maintained by the Contractor. The temporary lighting shall be constructed in accordance with local electrical code requirements. The temporary lighting shall be steady burning 60 watt, 120 volt lamps with molded waterproof lamp holders spaced at 25-foot centers maximum. All costs relating to pedestrian pathway paving, wood walks, overhead protection, maintenance, operating costs, and temporary pedestrian lighting shall be incidental to applicable adjacent items of work.

6-02.3(17)D Falsework Support Systems: Piling, Temporary Concrete Footings, Timber Mudsills, Manufactured Shoring Towers, Caps, and Posts

The Contractor shall support all falsework on either driven piling, temporary concrete footings, or timber mudsills. Temporary concrete footings shall be designed as reinforced concrete which may be either cast in place or precast. All components for a falsework support system shall be sized for the maximum design loads and allowable stresses described in the preceding sections.

The falsework drawings shall include a superstructure placing diagram showing the concrete placing sequence, direction of placements, and construction joint locations. When a sequence for placing concrete is shown in the Contract Plans or Specifications, no deviation will be permitted.

If the Plans call for piling or foundation shafts to support permanent structures, the Contractor may not use mudsills or temporary concrete footings for falsework support unless the underlying soil passes the settlement test described in this section.

Piling

When using piling to support the falsework, the Contractor's falsework plans shall specify the minimum required bearing and depth of penetration for the piling. Also, the falsework drawings shall show the maximum horizontal distance that the top of a falsework pile may be pulled in order to position it under its cap. The falsework plans shall show the maximum allowable deviation of the top of the pile, in its final position, from a vertical line through the point of fixity of the pile. The calculations shall account for pile stresses due to combined axial and flexural stress and secondary stresses.

Timber piling (untreated) shall be banded before driving. The following shall be identified in the falsework plans: lengths, minimum tip diameter, and expected diameter at ground line. The Contractor shall comply with the requirements of Sections 9-10.1 and 9-10.1(1). The maximum allowable load for timber piles shall be 45 tons. Steel piling shall be identified in the falsework plans. If steel pipe piling is used, the pipe diameter and wall thickness shall be identified in the falsework plans. Steel piling shall meet the requirements of Section 9-10.5. The formulas in Section 6-05.3(12) shall be used to determine the bearing capacity of the falsework piling. If the Engineer approves, the pile bearing capacity may instead be determined by test loading the piling to twice the falsework design load. The Contractor shall provide the Engineer an opportunity to witness these tests and provide a plan of the test and cross-sections showing the locations and elevations of the proposed tests to the Engineer for approval.

Temporary Concrete Footings and Timber Mudsills

Timber mudsills or temporary concrete footings may be used in place of driven piling, provided tests show that the soil can support twice the falsework design load and that the mudsill or temporary concrete footing will not settle more than $^1/_4$-inch when loaded with the design load. The tests shall be done at the falsework site, at the same elevation of the mudsill, and conducted under conditions representative of the actual site conditions. The acceptable tests for various soil types are:

1. **Granular Soil.** The Contractor shall conduct on-site tests according to AASHTO T 235. The Contractor shall provide the Engineer an opportunity to witness these tests and provide a plan of the test and cross-sections showing the locations and elevations of the proposed tests to the Engineer for approval.
2. **Fine Grained or Organic Soil.** The Contractor shall employ a Geotechnical Engineer to investigate the foundation soils and certify in writing that each mudsill or temporary footing will meet the load-settlement requirements described above. The allowable bearing capacities, elevations and locations of specific falsework mudsills shall be listed in the certification. Soils information used to determine the soil bearing capacity and settlement shall be submitted with the written certification to the Engineer for review and approval.

Timber mudsills or temporary concrete footings for falsework shall be designed to carry the loads imposed upon them without exceeding the estimated soil bearing capacity and specified maximum settlement. Where mudsills or temporary footings are used in the vicinity of permanent spread footings, the allowable mudsill bearing pressure shall be less than that of the permanent footings. This is because elevation difference, smaller bearing area, and the lack of surrounding overburden provides a lower bearing capacity than the permanent spread footings. The mudsills shall be designed for bearing capacities at the location that they are to be used. Timber mudsills or temporary concrete footings shall be designed as unyielding foundations under full design loads. The soil pressure bearing values assumed in the design of the falsework (normally not more than 3,000 pounds per sq. ft.) shall be shown in the falsework drawings. The minimum edge distances from the edge of the post or shoring tower leg to the edge or end of the mudsill member shall be shown in the falsework drawings. Timber mudsills and temporary concrete footings shall be designed such that member deflections do not exceed $^1/_4$-inch and that member allowable stresses are not exceeded.

Full cross-sectional views of all falsework on timber mudsills or temporary concrete footings to be placed in side slopes or above excavations shall be shown in the falsework drawings. Footings or mudsills which are stepped or placed above an excavation shall have all related geometry and slope stability items identified in the falsework plan. Details and calculations for any shoring system to support the embankment or excavation shall be included.

Mudsills or temporary concrete footings placed in benches in slopes shall be set back from the face of the slope one-half the mudsill or temporary concrete footing width, but not less than 1-foot 0-inches. The bench including the setback shall be level in its narrow dimension. Slopes between benches measured from the top of slope at one bench to the toe of slope at the next bench below shall be no steeper than $1^1/_2$ horizontal to 1 vertical.

Falsework shall be founded on a solid footing, safe against undermining, protected from softening, and capable of supporting the loads imposed. The preparation of the soil to receive the temporary footing is important to ensure that the falsework does not experience localized settlement that could result in falsework failure. In preparing the soil for a timber mudsill or temporary concrete footing, the Contractor shall:

1. Place it on dry soil that is either undisturbed or compacted to 95 percent of maximum density, as determined by the compaction control tests in Section 2-03.3(14)D performed by the Contractor and submitted to the Engineer for review;
2. Place mudsills or footings level with full contact bearing on the soil with no voids. Place each distribution plate or corbel member between the post or tower leg and the mudsill members such that there is full contact bearing;
3. Place grout or a compacted layer of fine material under the mudsill if it is supported by rock or coarse sand and gravel;
4. Provide the Engineer with a sample of any off-site material to be used under the mudsill;
5. Allow up to five working days for the Engineer's approval before using the off-site material; and
6. Provide erosion control measures to protect the soil of the mudsill or footing from undermining and softening.

Anticipated total settlements and incremental settlements of falsework and forms due to successive concrete placements shall be shown in the falsework plans. These shall include falsework footing settlement and joint take-up. Total anticipated settlements shall not exceed 1-inch including joint take-up. When using mudsills, the Contractor shall prepare for the possibility of reshoring with the use of such devices as screw jacks or hydraulic jacks and adjustment of wedge packs. The placing of concrete shall be discontinued if unanticipated settlement occurs, including settlements that deviate more than plus or minus $^3/_8$-inch from those indicated on the approved falsework drawing. Concrete placement shall not resume until corrective measures satisfactory to the Engineer are provided. If satisfactory corrective measures are not provided prior to initial set of the concrete in the affected area, placing of concrete shall be discontinued at a location determined by the Engineer. All unacceptable concrete shall be removed as determined by the Engineer.

Where the maximum leg load exceeds 30 kips, foundations for individual steel towers shall be designed and constructed to provide uniform settlement at each tower leg for all loading conditions.

Bents, Shoring Towers, Piling, Posts, and Caps

Plans for falsework bents or shoring tower systems, including manufactured tower systems shall have plan, cross-section, and elevation view scale drawings showing all geometry. Show in the falsework plans the proximity of falsework to utilities or any nearby structures including underground structures. The ground elevation, cross-slopes, relation of stringers to one another, and dimensions to posts or piling shall be shown in the falsework plans. Column, pile, or tower heights shall be indicated. Member sizes, wall thickness and diameter of steel pipe columns or piles shall be shown in the falsework plans. Location of wedges, minimum bearing area and type of wedge material shall be identified in the falsework plans. Bracing size, location, material and all connections shall be described in the falsework plans.

The relationship of the falsework bents or shoring tower systems to the permanent structure's pier and footing shall be shown. Load paths shall be as direct as possible. Loads shall be applied through the shear centers of all members to avoid torsion and buckling conditions. Where loads cause twisting, biaxial bending, or axial loading with bending, the affected members shall be designed for combined stresses and stability.

Posts or columns shall be constructed plumb with tops and bottoms carefully cut to provide full end bearing. Caps shall be installed at all bents supported by posts or piling unless approved falsework plans specifically permit otherwise. Caps shall be fastened to the piling or posts. The falsework shall be capable of supporting non uniform or localized loading without adverse effect. For example, the loading of cantilevered ends of stringers or caps shall not cause a condition of instability in the adjacent unloaded members.

Timber posts and piling shall be fastened to the caps and mudsills by through-bolted connections, drift pins, or other approved connections. The minimum diameter of round timber posts shall be shown in the falsework plans. Timber caps and timber mudsills shall be checked for crushing from columns or piling under maximum load.

Steel posts and piling shall be welded or bolted to the caps, and shall be bolted or welded to the foundation. Steel members shall be checked for buckling, web yielding, and web crippling.

Wedges shall be used to permit formwork to be taken up and released uniformly. Wedges shall be oak or close-grained Douglas fir. Cedar wedges or shims shall not be used anywhere in a falsework or forming system. Wedges shall be used at the top or bottom of shores, but not at both top and bottom. After the final adjustment of the shore elevation is complete, the wedges shall be fastened securely to the sill or cap beam. Only one set of wedges (with one optional block) shall be used at one location. Screw jacks (or other approved devices) shall be used under arches to allow incremental release of the falsework.

Sand jacks may be used to support falsework and are used for falsework lowering only. Sand jacks shall be constructed of steel with snug fitting steel or concrete pistons. Sand jacks shall be filled with dry sand and the jack protected from moisture throughout its use. They shall be designed and installed in such a way to prevent the unintentional migration or loss of sand. All sand jacks shall be tested per Section 6-02.3(17)G.

When falsework is over or adjacent to roadways or railroads, all details of the falsework system which contribute to the horizontal stability and resistance to impact shall be installed at the time each element of the falsework is erected and shall remain in place until the falsework is removed. For other requirements see Section 6-02.3(17)C.

Transverse construction joints in the superstructure shall be supported by falsework at the joint location. The falsework shall be constructed in such a manner that subsequent pours will not produce additional stresses in the concrete already in place.

Manufactured Shoring Tower Systems and Devices

Manufactured proprietary shoring tower systems shall be identified in the falsework plans by make and model and safe working load capacity per leg. The safe working load for shoring tower systems shall be based upon a minimum $2^1/_2$ to 1 factor of safety.

The safe working load capacity, anticipated deflection (or settlement), make and model shall be identified in the falsework plans for manufactured devices such as: single shores, overhang brackets, support bracket and jack assemblies, friction collars and clamps, hangers, saddles, and sand jacks. The safe working load for shop manufactured devices shall be based on a minimum ultimate strength safety factor of 2 to 1. The safe working load for field fabricated devices and all single shores shall be based on a minimum ultimate strength safety factor of 3 to 1.

The safe working load of all devices shall not be exceeded. The design loads shall be as defined by Section 6-02.3(17)A. The maximum allowable free end deflection of deck overhang brackets under working loads applied shall not exceed $^3/_{16}$-inch measured at the edge of the concrete slab regardless of the fact that the deflection may be compensated for by pre-cambering or of setting the elevations high. The Contractor shall comply with all manufacturer's specifications; including those relating to bolt torque, placing washers under nuts and bolt heads, cleaning and oiling of parts, and the reuse of material. Devices which are deteriorated, bent, warped, or have poorly fitted connections or welds, shall not be installed.

Shoring tower or device capacity as shown in catalogs or brochures published by the manufacturer shall be considered as the maximum load which the shoring is able to safely support under ideal conditions. These maximum values shall be reduced for adverse loading conditions; such as horizontal loads, eccentricity due to unbalanced spans or placing sequence, and uneven foundation settlement.

Depending on load-carrying capacity, steel shoring systems are classified as pipe-frame systems, intermediate strength systems, and heavy-duty systems. The two types of pipe-frame shoring base frames in general use are the ladder type and the cross-braced type. In the ladder type, frame rigidity is provided by horizontal struts between the vertical legs, whereas in the cross-braced type rigidity is provided by diagonal cross-bracing between the legs.

Copies of catalog data and/or other technical data shall be furnished with the falsework plans to verify the load-carrying capacity, deflection, and manufacturers installation requirements of any manufactured product or device proposed for use. Upon request by the Engineer, the Contractor shall furnish manufacturer certified test reports and results showing load capacity, deflection, test installation conditions, and identify associated components and hardware for shoring tower systems or other devices. In addition to manufacturer's requirements, the criteria shown in the following sections for manufactured proprietary shoring tower systems and devices shall be complied with when preparing falsework plans, calculations, and installing these shoring tower systems and devices as falsework.

Alternative criteria and/or systems may be approved if a written statement on the manufacturer's letter head, signed by the shoring or device manufacturer (not signed by a material supplier or the Contractor) is submitted to the Engineer for approval and addresses the following:

1. Identity of the specific Contract on which the alternative criteria and/or system will apply;
2. Description of the alternative criteria and/or system;
3. Technical data and test reports;
4. The conditions under which the particular alternative criteria may be followed;
5. That a design based on the alternative criteria will not overstress or over deflect any shoring component or device nor reduce the required safety factor.

In any case where the falsework drawings detail a manufactured product and the manufacturer's safe working load, load versus deflection curves, factor of safety, and installation requirements cannot be found in any catalog, the Engineer may require load testing per Section 6-02.3(17)G to verify the safe working load and deflection characteristics.

Tower leg loads shall not exceed the limiting values under any loading condition or sequence. Frame extensions and any reduced capacity shall be shown in the falsework plans. Screw jacks shall fit tight in the leg assemblies without wobble. Screw jacks shall be plumb and straight. Shoring towers shall be installed plumb, and load distribution beams shall be arranged such that vertical loads are distributed to all legs for all successive concrete placements. There shall be no eccentric loads on shoring tower heads unless the heads have been designed for such loading. Shoring towers shall remain square or rectangular in plan view and shall not be skewed. There shall be no interchanging of parts from one manufactured shoring system to another. Bent or faulty components shall not be used.

For manufactured shoring towers that allow ganging of frames, the number of ganged frames shall be limited to one frame per opposing side of a tower, and the total number of legs per ganged tower shall not exceed eight legs. Ganged frames shall be installed per the manufacturer's published standards using the manufacturer's components. Other gang arrangements shall not be used.

For manufactured steel shoring tower systems, the contractor shall have bracing designed and installed for horizontal loads and falsework overturning per Section 6-02.3(17)A. Minimum bracing criteria and allowable leg loads are described in the following paragraphs.

All shoring tower systems and bracing shall be thoroughly inspected by the Contractor for plumb vertical support members, secure connections, and straight bracing members immediately prior to, at intervals during, and immediately after every concrete placement. For manufactured shoring tower systems, the maximum allowable deviation from the vertical is $^{1}/_{8}$-inch in 3-feet. If this tolerance is exceeded, concrete shall not be placed until adjustments have brought the shoring towers within the acceptable tolerance.

Cross-Braced Type Base Frames

The maximum allowable load per leg for cross-braced type base frame shoring is limited by the height of the extension frame and the type of screw jack (swivel or fixed head) used at the top of the frame. The maximum load on one leg of a frame shall not exceed four times the load on the other leg under any given loading condition or sequence. The maximum load on one of the two frames making up a tower shall not exceed four times the load on the opposite frame under any given loading condition or sequence. If swivel-head screw jacks are used, the allowable leg loads shall not exceed that shown in the following table:

Maximum Allowable Leg Load in Pounds				
Extension Frame Height	**2′-0"**	**3′-0"**	**4′-0"**	**5′-0"**
Screw height 12" or less	11,000	11,000	10,000	9,400
Screw height exceeds 12"	8,200	8,200	8,000	7,800

If fixed-head screw jacks are used at the top of the extension frame, the maximum allowable load per leg shall be 11,000 pounds for all extension frame heights up to 5-feet with screw jack height extensions of 12-inches or less. Fixed-head screw jacks exceeding 12-inches shall use the values in the table above. Screw jack extensions shall not exceed the manufacturer's published recommendations. Extension frames shall be braced. Side cross-braces are required for extension heights up to 2-feet 0-inches. Both side and end cross-braces are required from over 2-feet 0-inches to 5-feet 0-inches extension heights.

Supplemental bracing shall be installed on shoring towers 20-feet or more in height and shall connect rows of towers to each other so rows of frames are continuously cross-braced in one plane. Supplemental bracing shall be installed as follows:

1. In the transverse direction (the direction parallel to the frame) one horizontal brace and one diagonal brace shall be attached to each tower face, for every three frames of shoring height, including an extension frame if used. The lowest horizontal brace shall be located near the top of the third tower frame, and any additional horizontal braces spaced no farther than three frames apart. The diagonal braces shall be located on opposite tower faces, and shall run in opposite directions across the plane of the tower row.
2. In the longitudinal direction (the direction perpendicular to the frames), when shoring height is four frames or more, a horizontal brace shall be installed on one face of each tower, with the lowest brace located no higher than the top of the fourth frame and any additional horizontal braces spaced no farther than four frames apart. When shoring height is six frames or more, diagonal cross-bracing shall be installed in the longitudinal direction similar to the transverse direction.
3. When roadway grade, soffit profile, or superelevation exceeds 4 percent slope for any height of shoring tower, a continuous brace parallel to the slope shall be attached to each frame extension or screw jack of the tower within 6-inches of the top. These braces shall be in addition to bracing previously described.

The bracing shall be fastened securely to each frame leg and shall be located within 6-inches of the frame member intersections. The ends of diagonal braces shall not be attached to shoring frames at locations where towers have little or no load. Diagonal brace ends shall be attached to tower frames near the top and bottom at locations where significant gravity load is maintained throughout all construction sequences, such as directly below box girder outside webs, thus precluding lift-off due to the vertical component of the brace reaction. Supplemental bracing shall be shown in the falsework drawings. The connection details, including the method of connection and exact location of the connecting devices, shall be in accordance with the manufacturer's recommendations and shall be shown in the falsework drawings.

Ladder Type Base Frames

Ladder type base frame shoring shall be limited to the following maximum loads and conditions, regardless of any conflicting information which may be found in manufacturer's catalogs or brochures:

1. If the shoring system consists of a single tier of braced base frames, leg loads shall not exceed 10,000 pounds.
2. If the shoring system consists of two or three tiers of base frames, leg loads shall not exceed 7,500 pounds.
3. If an extension staff is used, the maximum allowable leg load shall be reduced to 6,000 pounds.
4. The maximum load on one leg of a frame shall not exceed four times the load on the other leg under any given loading condition or sequence. The maximum load on one of the two frames making up a tower shall not exceed four times the load on the opposite frame under any given loading condition or sequence.

Maximum allowable leg loads as shown above shall apply when fixed-head screw jacks are used, or when swivel-head jacks are used at either the top or bottom of the tower. A screw jack extension shall not exceed 12-inches. Swivel-head screw jacks shall not be used at both the top and bottom of ladder-type frames. For any combination of ladder-type base frames or base frames with staff extensions, the total height of the shoring shall not exceed 20-feet, including screw jack extensions.

When roadway grade, soffit profile, or superelevation exceeds 4 percent slope for heights of shoring towers 20-feet or less, a continuous brace parallel to the slope shall be attached to each staff extension or screw jack of the tower within 6-inches of the top. These braces shall be attached per conditions described previously for cross-braced frames.

Intermediate Strength Shoring

Steel shoring, consisting of cross-braced tubular members capable of carrying up to 25 kips per tower leg, is considered intermediate strength shoring. The use of a 25-kip type falsework shoring system shall meet the following conditions and limitations:

1. If swivel-head screw jacks are used at either the top or bottom of the tower, the maximum allowable load shall be reduced to 20 kips per tower leg.
2. The screw-jack extensions shall not exceed 14-inches.
3. Extension frames shall be braced. Side cross-braces are required for all extension-frame heights. In addition, end cross-braces (braces across the face of the extension frame) shall be provided for extension frame heights of 3-feet or more.
4. The maximum load on one leg of a frame, or on one frame of a tower, shall not exceed four times the load on the opposite leg or frame under any given loading condition or sequence.
5. Shoring towers 20-feet or more in height shall have supplemental bracing installed in accordance with the criteria for bracing "Cross-braced Type Base Frames," except that no supplemental bracing will be required in the longitudinal direction (the direction perpendicular to the frame).
6. When roadway grade, soffit profile, or superelevation exceeds 4 percent slope for any height of shoring tower, a continuous brace parallel to the slope shall be attached to each frame extension or screw jack of the tower within 6-inches of the top. These braces shall be in addition to bracing required in item 5.

The use of 25-kip shoring, when designed and erected in conformance with the above criteria, is acceptable for tower heights up to five frames plus a fully-extended extension frame plus the maximum allowable screw-jack adjustment. For any proposed use exceeding this limiting height, the Contractor shall furnish a statement signed by the shoring manufacturer covering the specific installation. The statement shall provide assurance that the shoring will carry the loads to be imposed without overstressing any shoring component or reducing the required safety factor.

Heavy-Duty Shoring Systems

Shoring capable of carrying up to 100 kips per tower leg is considered heavy duty shoring. The following criteria applies to these systems.

If tower legs, including any extension unit, are utilized as single-post shores braced in one direction only, the shores shall be analyzed as individual steel columns.

If the total height of the shoring does not exceed the height of a single tower unit, including any extension unit, and if both the base and extension units are fully braced in both directions in accordance with the manufacturer's recommendations, individual tower legs may be considered as capable of carrying the safe working load recommended by the manufacturer without regard to the load on adjacent legs.

If the shoring consists of two or more units stacked one above the other, either with or without an extension unit, the differential leg loading within a given tower unit shall not exceed the following limitations:

Differential Leg Loading

Maximum load on any leg in the tower unit	Maximum to Minimum load ratio
10 kips or less	10 to 1
10 kips to 50 kips	6 to 1
50 kips to 75 kips	5 to 1
75 kips or more	4 to 1

A complete stress analysis of steel beams used as continuous caps over two or more tower units shall be performed to determine the effect of continuity on tower leg loads. Resulting moment shear shall be added to or subtracted from the simple beam reaction to obtain the actual leg load and may produce a significant load differential.

Heavy-duty shoring shall be diagonally braced or otherwise externally supported at the top unless the towers are stable against overturning as defined in Section 6-02.3(17)A. When designing external bracing, including cable bracing, attention shall be given to the bracing connection to the falsework. Connections shall be designed to transfer horizontal and vertical forces from the falsework to the bracing system without overstressing any tower component. All external bracing, attachment locations, and connection details shall be shown in the falsework plans.

6-02.3(17)E Stringers, Beams, Joists, Roadway Slab Support, and Deck Overhangs

All stringers, beams, joists, and roadway slab support shall be designed for the design loads, deflections, and allowable stresses described in the preceding Sections 6-02.3(17)A, B, and C and for the following conditions.

At points of support, stringers, beams, joists, and trusses shall be restrained against rotation about their longitudinal axis. The effect of biaxial bending shall be investigated in all cases where falsework beams are not set plumb and the structure cross-slope exceeds 3 percent.

For box girder and T-beam bridges, the centerline of falsework beams or stringers shall be located within 2-feet of the bridge girder stems and preferably directly under the stems or webs. Stringers supporting formwork for concrete box girder and T-beam slab overhangs shall be stiff enough so that the differential deflection due to the roadway slab pour is no more than $^{3}/_{16}$-inch between the outside edge of the roadway slab and the exterior web even if camber strips can compensate for the deflection.

Friction shall not be relied upon for lateral stability of beams or stringers. If the compression flange of a beam is not laterally restrained, the allowable bending stress shall be reduced to prevent flange buckling. If flange restraint is provided and since it is impossible to predict the direction in which a compression flange will buckle, positive restraint shall be provided in both directions. Flange restraint shall be designed for a minimum load of two percent of the calculated compression force in the beam flange at the point under consideration.

Camber strips shall be used to compensate for falsework take-up and deflection, vertical alignment, and the anticipated structure dead load deflection shown in the camber diagram in the Contract Plans. Camber is the adjustment to the profile of a load-supporting beam or stringer so that the completed structure will have the lines and grades shown in the Plans. The dead load camber diagram shown in the Contract Plans is the predicted structure dead load deflection due to self mass. This dead load camber shall be increased by:

1. Amount of anticipated falsework take up;
2. Anticipated deflection of the falsework beam or stringer under the actual load imposed; and
3. Any vertical curve compensation.

Camber strips shall be fastened by nailing to the top of wood members, or by clamping or banding in the case of steel members. Camber strips shall have sufficient contact bearing area to prevent crushing under total load. As a general rule, camber strips are not required unless the total camber adjustment exceeds $^{1}/_{4}$-inch for exterior falsework stringers and $^{1}/_{2}$-inch for interior stringers.

On concrete box girder structures, the forms supporting the roadway slab shall rest on ledgers or similar supports and shall not be supported from the bottom slab except as provided below. The form supports shall be fastened within 18-inches of the top of the web walls, producing a clear span between web walls. The roadway slab forms may be supported or posted from the bottom slab if the following conditions are met:

1. Permanent access, shown in the Contract Plans, is provided to the cells, and the centerline to centerline distance between web walls is greater than 10-feet;
2. Falsework stringers designed for total load, stresses and deflections per Section 6-02.3(17)A and B are located directly below each row of posts;
3. Posts have adequate lateral restraint; and
4. All forms (including the roadway deck forms), posts, and bracing are completely removed.

The falsework and forms on concrete box girder structures supporting a sloping web and deck overhang shall consist of a lateral support system which is designed to resist all rotational forces acting on the stem, including those caused by the placement of deck slab concrete, roadway deck formwork mass, finishing machine, and other live loads. Stem reinforcing steel shall not be stressed by the construction of the roadway deck slab placement. Overhang brackets shall not be used for the support of roadway slab forms from sloping web concrete box girder bridges.

Deck slab forms between girders or webs shall be constructed such that there is no differential settlement relative to the girders. The support systems for form panels supporting concrete deck slabs and overhangs on girder bridges (such as steel plate girders and prestressed girders) shall be designed as falsework. Falsework supporting deck slabs and overhangs on girder bridges shall be supported directly by the girders so that there will be no differential settlement between the girders and the deck forms during placement of deck concrete.

6-02.3(17)F Bracing

All falsework bracing systems shall be designed to resist the horizontal design load in all directions with the falsework in either the loaded or unloaded condition. All bracing, connection details, specific locations of connections, and hardware used shall be shown in the falsework plans. Falsework diagonal bracing shall be thoroughly analyzed with particular attention given to the connections. The allowable stresses in the diagonal braces may be controlled by the joint strength or the compression stability of the diagonal. Timber bracing for timber falsework bents shall have connections designed per Section 6-02.3(17)I. Any damaged cross-bracing, such as split timber members shall be replaced. Steel strapping shall avoid making sharp angles or right-angle bends. A means of preventing accidental loss of tension shall be provided for steel strapping. See Sections 6-02.3(17)A, B, and C for design loads and allowable stresses.

Bracing shall not be attached to concrete traffic barrier, guardrail posts, or guardrail.

To prevent falsework beam or stringer compression flange buckling, cross-bracing members and connections shall be designed to carry tension as well as compression. All components, connection details and specific locations shall be shown in the falsework plans. Bracing, blocking, struts, and ties required for positive lateral restraint of beam flanges shall be installed at right angles to the beam in plan view. If possible, bracing in adjacent bays shall be set in the same transverse plane. However, if because of skew or other considerations, it is necessary to offset the bracing in adjacent bays, the offset distance shall not exceed twice the depth of the beam.

All falsework and bracing shall be inspected by the Contractor for plumbness of vertical support members, secure connections, tight cables, and straight bracing members immediately prior to, during, and immediately after every concrete placement.

Bracing shall be provided to withstand all imposed loads during erection of the falsework and all phases of construction for falsework adjacent to any roadway, sidewalk, or railroad track which is open to the public. All details of the falsework system which contribute to horizontal stability and resistance to impact, including the bolts in bracing, shall be installed at the time each element of the falsework is erected and shall remain in place until the falsework is removed. The falsework plans shall show provisions for any supplemental bracing or methods to be used to conform to this requirement during each phase of erection and removal. Wind loads shall be included in the design of such bracing or methods. Loads, connections, and materials for falsework adjacent to roadways, shall also be in accordance with Section 6-02.3(17)C.

Cable or Tension Bracing Systems

When cables, wire rope, steel rod, or other types of tension bracing members are used as external bracing to resist horizontal forces, or as temporary bracing to support bents while falsework is being erected or removed adjacent to traffic, all elements of the bracing system shall be shown in the falsework plans. Bracing shall not be attached to concrete traffic barrier, guardrail posts, or guardrail. Any damaged bracing, such as frayed and kinked guying systems shall be replaced. Wire rope shall avoid making sharp angles or right-angle bends and a means of preventing accidental loss of tension shall be provided. The following information shall be submitted to the Engineer for approval:

1. Cable diameter, rod, or tension member size, and allowable working load.
2. Location and method of attaching the cable, rod, or tension member to the falsework. The connecting device shall be designed to transfer both horizontal and vertical forces to the cable without overstressing any falsework component.
3. The type of cable connectors or fastening devices (such as U-bolt clips, plate clamps, etc.) to be used and the efficiency factor for each type. If cables are to be spliced, the splicing method shall be shown.
4. Method of tightening cables, rods, or tension members after installation if tightening is necessary to ensure their effectiveness. Method of preventing accidental loosening.
5. Anchorage details, including the size and mass of concrete anchor blocks, the assumed coefficient of friction for surface anchorages, and the assumed lateral soil bearing capacity for buried anchorages.
6. Method of pre-stretching or preloading cable or tension members.
7. Determination of the potential stretch or elongation of the tension member under the design load and if the resulting lateral deflection will cause excessive secondary stresses in the falsework.

Copies of manufacturer’s catalog or brochure showing technical data pertaining to the type of cable to be used shall be furnished with the falsework plans. Technical data shall include the cable diameter, the number of strands and the number of wires per strand, ultimate breaking strength or recommended safe working strength, and any other information as may be needed to identify the cable.

In the absence of sufficient technical data to identify the cable, or if it is old and obviously worn, the Contractor shall perform cable breaking tests to establish the safe working load for each reel of cable furnished. For static guy cable the minimum factor of safety shall be 3 to 1. The Contractor shall provide the Engineer an opportunity to witness these tests.

When cable bracing is used to prevent the overturning of heavy-duty shoring, attention shall be given to the connections by which forces are transferred from the shoring to the cables. Cable restraint shall be designed to act through the cap system to prevent the inadvertent application of forces which the shoring is not designed to withstand. Cables shall not be attached to any tower component.

Cable splices made by lapping and clipping with “Crosby” type clamps shall not be used. Other splicing methods may be used; however, at each location where the cable is spliced, cable strength shall be verified by a load test.

When cables are used as external bracing to resist overturning of a falsework system, the horizontal load to be carried by the cables shall be calculated as follows:

1. When used with heavy-duty shoring systems, cables shall be designed to resist the difference between 1.25 times the total overturning moment and the resistance to overturning provided by the individual falsework towers.
2. When used with pipe-frame shoring systems where supplemental bracing is required, cables shall be designed to resist the difference between 1.25 times the total overturning moment and the resistance to overturning provided by the shoring system as a whole.
3. When used as external bracing to prevent overturning of all other types of falsework, including temporary support during erection and removal of falsework at traffic openings, cables shall be designed to resist 1.25 times the total overturning moment.

The maximum allowable cable design load shall be determined using the following criteria:

1. If the cable is new, or is in uniformly good condition, and if it can be identified by reference to a manufacturer's catalog or other technical publication, the allowable load shall be the ultimate strength of the cable as specified by the manufacturer, multiplied by the efficiency of the cable connector, and divided by a safety factor of 3 (i.e., safe working load = breaking strength x connector efficiency/safety factor).
2. If the cable is used but still in serviceable condition, or is new or nearly new but cannot be found in a manufacturer's catalog, the Contractor shall perform load breaking tests. In this case, the cable design load shall not exceed the breaking strength, as determined by the load test, multiplied by the connector efficiency factor, and divided by a safety factor of 3.
3. If the cable is used and still in serviceable condition, or is a new or nearly new cable which cannot be identified, and if load breaking tests are not performed, the cable design load shall not exceed the safe working load shown in the wire rope capacities table multiplied by the cable connector efficiency.

   Cable connectors shall be designed in accordance with criteria shown in the following tables "Efficiency of Wire Rope Connections" and "Applying Wire Rope Clips." Cable safe working loads are provided in table "Wire Rope Capacities."

Efficiency of Wire Rope Connections (As compared to Safe Loads on Wire Rope)	
Type of Connection	**Connector Efficiency**
Wire Rope	100%
Sockets — Zink Type	100%
Wedge Sockets	70%
Clips — Crosby Type With Thimble	80%
Knot and Clip (Contractors Knot)	50%
Plate Clamp — Three Bolt Type With Thimble	80%
Spliced Eye and Thimble:	
1/4" and smaller	100%
3/8" to 3/4"	95%
7/8" to 1"	88%
1 1/8" to 1 1/2"	82%
1 5/8" to 2"	75%
2 1/8" and larger	70%

Wire Rope Capacities
Safe Load in Pounds for New Plow Steel Hoisting Rope
6 Strands of 19 Wires, Hemp Center
(Safety Factor of 6)

Diameter Inches	Weight Lbs./Ft.	Safe Load Lbs.
1/4	0.10	1,050
5/16	0.16	1,500
3/8	0.23	2,250
7/16	0.31	3,070
1/2	0.40	4,030
9/16	0.51	4,840
5/8	0.63	6,330
3/4	0.95	7,930
7/8	1.29	10,730
1	1.60	15,000
1 1/8	2.03	18,600
1 1/4	2.50	23,000
1 3/8	3.03	25,900
1 1/2	3.60	30,700
1 5/8	4.23	35,700
1 3/4	4.90	41,300

Applying Wire Rope Clips

The only correct method of attaching U-bolt wire rope clips to rope ends is to place the base (saddle) of the clip against the live end of the rope, while the "U" of the bolt presses against the dead end.

The clips are usually spaced about six rope diameters apart to give adequate holding power. A wire-rope thimble shall be used in the loop eye to prevent kinking when wire rope clips are used. The correct number of clips for safe application, and spacing distances, are shown below:

Number of Clips and Spacing for Safe Application

Improved Plow Steel Rope Diameter Inches	Number of Clips: Drop Forged	Number of Clips: Other Material	Minimum Spacing (Inches)
3/8	2	3	3
1/2	3	4	3 1/2
5/8	3	4	4
3/4	4	5	4 1/2
7/8	4	5	5 1/4
1	5	6	6
1 1/8	6	6	6 3/4
1 1/4	6	7	7 1/2
1 3/8	7	7	8 1/4
1 1/2	7	8	9

Anchor Blocks

Concrete anchor blocks and connections used to resist forces from external bracing shall be shown in the falsework plans. Concrete anchor blocks shall be proportioned to resist both sliding and overturning. When designing anchor block stability, the mass of the anchor block shall be reduced by the vertical component of the cable or brace tension to obtain the net or effective mass to be used in the anchorage computations.
The coefficient of friction assumed in the design shall not exceed the following:

	Friction Coefficient
Anchor block set on sand	0.40
Anchor block set on clay	0.50
Anchor block set on gravel	0.60
Anchor block set on pavement	0.60

Note: Multiply the friction coefficient by 0.67 if it is likely the supporting material is wet or will become wet during the construction period.

The method of connecting the cable or brace to the anchor block is part of the anchor block design. The connection shall be designed to resist both horizontal and vertical forces.

Temporary Bracing for Bridge Girders

Bridge girders (such as steel plate girders and prestressed girders) shall be braced and tied to resist forces that would cause rotation or torsion in the girders caused by the placing of concrete for diaphragms or the deck. These conditions also apply to bridge widenings and stage constructed bridges where construction sequences can cause rotation or torsion in the girders. Falsework support brackets or braces shall not be welded to structural steel members or reinforcing steel.

On prestressed girder spans, the Contractor shall install cross-bracing between girders at each end and midspan to prevent lateral movement or rotation. This bracing shall be placed prior to the release of the girders from the erection equipment. The bracing shall not be removed until the diaphragms or the deck have been placed and cured for a minimum of 24 hours.

When deck overhang or the distance from the centerline of the exterior girder (or outside girder of a staged construction) to the near edge of the roadway slab on a prestressed girder span exceeds the distances listed in the table below, the Contractor shall provide extra bracing for the exterior girder at the midpoint between diaphragms (or at more frequent intervals). This bracing shall include: (1) a cross-tie connecting the top flange of each exterior girder with its counterpart on the other side, and (2) braces between the bottom flanges and top flanges of all girders.

Girder Series	Distance in Inches
W42G	30
W50G	42
W58G	63
W74G	66
Prestressed concrete tub girders with webs with flanges	30
WF42G, WF50G, WF58G, WF74G, W83G, and W95G	70
W32BTG, W38BTG, and W62BTG	70
WF74PTG, W83PTG, and W95PTG	70

If a concrete finishing machine is supported at the outside edge of the slab, the Contractor shall account for its added mass in the design of bracing.

Roadway deck forming systems may require bracing or ties between girders for the girder to adequately support the form loading. When braces, struts, or ties are required, they shall be designed and detailed by the Contractor and shall be shown in the falsework/formwork plans submitted to the Engineer for approval. These braces, struts, and ties shall be furnished and installed by the Contractor at no additional cost to the Contracting Agency.

6-02.3(17)G Testing Falsework Devices

The Contractor shall establish the load capacity and deflection (or settlement) of all friction collars and clamps, brackets, hangers, saddles, sand jacks, and similar devices utilizing a recognized independent testing laboratory approved by the Engineer. Laboratory tests shall use the same materials and design that will be used on the project. Test loads shall be applied to the device in the same manner that the device will experience loading on the project. Any bolts or threaded rods used with the device shall be identified as to diameter, length, type, grade, and torque. Any wedges, blocks, or shims used with the device on the project shall also be tested with the device. Any adjustable jack system used as a part of a device shall be tested with the device and shall have its maximum safe working extended height identified. Devices shall not be tested in contact with the permanent structure. Independent members with the same properties as the permanent structure shall be used to test device connections.

At least fourteen (14) days prior to the test, the Contractor shall submit a test procedure and scale drawing for the Engineer's approval showing how the device will be tested and how data will be collected. The Contractor shall provide the Engineer an opportunity to witness these tests.

The approved independent testing laboratory shall provide a certified test report which shall be signed and dated. The test report shall clearly identify the device tested including trademarks and model numbers; identify all parts and materials used, including grade of steel, or lumber, member section dimensions; location, size, and the maximum tested extended height of any adjustable jacks; indicate condition of materials used in the device; indicate the size, length and location of all welds; indicate how much torque was used with all bolts and threaded rods. The report shall describe how the device was tested, report the results of the test, provide a scale drawing of the device showing the location(s) of where deflections or settlements were measured, and show where load was applied. Deflections or settlements shall be measured at load increments and the results shall be clearly graphed and labeled. Prior to installation of falsework devices named in this section, the Contractor shall submit the certified test reports to the Engineer for review and approval.

The safe working load for shop manufactured devices named in this section shall be derived by dividing the ultimate strength by a safety factor of 2.0. The safe working load for field fabricated or field modified devices (including the use of timber blocks or wedges with the device) shall be determined by dividing the ultimate strength by a safety factor of 3.0. Working load shall include masses of all successive concrete placements, falsework, forms, all load transfer that takes place during post-tensioning, and any live loads; such as workers, roadway finishing machines, and concrete delivery systems. The maximum allowable free end deflection of deck overhang brackets with combined dead and live working loads applied shall be $^3/_{16}$-inch even though deflection may be compensated for by pre-cambering or setting the elevations high. The Contractor shall comply with all manufacturer's specifications; including those relating to bolt torque, cleaning and oiling of parts, and the reuse of material. Devices which are deteriorated, bent, warped or have poorly fitted connections or welds, shall not be installed.

6-02.3(17)H Formwork Accessories

Formwork accessories such as form ties, form anchors, form hangers, anchoring inserts, and similar hardware shall be specifically identified in the formwork plans including the name and size of the hardware, manufacturer, safe working load, and factor of safety. The grade of steel shall also be indicated for threaded rods, coil rods, and similar hardware. Wire form ties and taper ties shall not be used. Welding or clamping formwork accessories to Contract Plan reinforcing steel will not be allowed. Driven types of anchorages for fastening forms or form supports to concrete, and Contractor fabricated "J" hooks shall not be used. Field drilling of holes in prestressed girders is not allowed.

The following table from ACI 347R-88 provides minimum safety factors for formwork accessories. The hardware proposed shall meet these minimum ultimate strength requirements or the manufacturer's minimum requirements, whichever provides the greater factor of safety. The Contractor shall attach copies of the manufacturer's catalog cuts and/or test data of hardware proposed, to the formwork plans and submit the falsework and formwork plans and calculations for review and approval per Section 6-02.3(16). In situations where catalog cuts and/or test data are not available, testing shall be performed in accordance with Section 6-02.3(17)G.

Minimum Safety Factors of Formwork Accessories*

Accessory	Safety Factor	Type of Construction
Form Tie	2.0	All applications.
Form Anchor	2.0	Formwork supporting form mass and concrete pressures only.
Form Anchor	3.0	Formwork supporting masses of forms, concrete, construction live loads, and impact.
Form Hangers	2.0	All applications.
Anchoring Inserts	2.0	Placed in previous opposing concrete placement to act as an anchor for form tie.

*Safety factors are based on ultimate strength of the formwork accessory.

The bearing area of external holding devices shall be adequate to prevent excessive bearing stress on form lumber. Form ties and form hangers shall be arranged symmetrically on the supporting members to minimize twisting or rotation of the members. Form tie elongation shall not exceed the allowable deflection of the wale or member that it supports. Inserts, bolts, coil rods, and other fasteners shall be analyzed and designed for appropriately combined bending, shear, torsion, and tension stresses. The formwork shall not be attached to Contract Plan rebar or rebar cages. However, the Contractor may install additional reinforcing steel for formwork anchorage.

Frictional resistance shall not be considered as contributing to the stability of any connection or connecting device, except those designed as friction connectors such as U-bolt friction-type connectors.

Form anchors and anchoring inserts shall be designed considering concrete strength at time of loading, available embedment, location in the member, and any other factors affecting their working strength, and shall be installed in concrete per the manufacturer's published requirements. Form anchors and anchoring inserts embedded in previous concrete placements shall not be loaded until the concrete has reached the required design strength. The required design strength of concrete for loading of an anchor shall be shown in the formwork drawing if it is assumed that the anchor will be loaded before the concrete has reached its 28 day strength.

Installation of permanent concrete inserts, such as form ties hangers, or embedded anchor assemblies, shall permit removal of all metal to at least 1/2-inch below the concrete surface. Holes shall be patched in accordance with Section 6-02.3(14). During removal of the outer unit, the bond between the concrete and the inner unit or rod shall not be broken.

6-02.3(17)I Timber Connections

Timber connections shall be designed in accordance with the methods, stresses, and loads allowed in the Timber Construction Manual, Third Edition by the American Institute of Timber Construction (AITC). Timber falsework and formwork connections shall be designed using wet condition stresses for all installations West of the Cascade Range crest line and by criteria provided in the following sections. Frictional resistance shall not be considered as contributing to the stability of any timber connection.

Bolted Connections

Tabulated values in the AITC Timber Construction Manual-Third Edition are based on square posts. For a round post or pile, the main member thickness shall be the side of a square post having the same cross-sectional area as the round post used.

The AITC Table 6.20 for Douglas Fir-Larch bolt Group 3 and for Hem-Fir bolt Group 8 show design values for bolts to be used when the load is applied either parallel or perpendicular to the direction of the wood grain. When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value for the main member shall be obtained from the Hankinson formula shown in the AITC manual.

Design values in the AITC Table 6.20 apply only to three member joints (bolt in double-shear) in which the side members are each $^1/_2$ the thickness of the main member. This joint configuration is not typical of bridge falsework where side members are usually much smaller than main members. For two member joints (single shear bolt condition), the AITC Table 6.20 values shall be adjusted by a single shear load factor as follows:

1. 0.75 for installations East of the Cascade Range crest line, except as shown in item 3 below;
2. 0.50 for installations West of the Cascade Range crest line; and
3. 0.50 for load acting at an angle to the bolt axis, as is the case with longitudinal bracing when falsework bents are skewed.

Except for connections in falsework adjacent to or over railroads or roadways, threaded rods and coil rods may be used in place of bolts of the same diameter with no reduction in the tabulated values. At openings for roadways and railroads, all connections shall be bolted using $^5/_8$-inch diameter or larger through bolts.

Bolt holes shall be a minimum $^1/_{32}$-inch to a maximum $^1/_8$-inch larger than the bolt diameter. A washer not less than a standard cut washer shall be installed between the wood and the bolt head and between the wood and the nut to distribute the bearing stress under the bolt head and nut and to avoid crushing the fibers. In lieu of standard cut washers, metal plates or straps with dimensions at least equal to that of a standard cut washer may be substituted.

When steel bars or shapes are used as diagonal bracing, the tabulated design values shown in AITC Table 6.20 for the main members loaded parallel to grain (P value) are increased 75 percent for joints made with bolts $^1/_2$-inch or less in diameter, 25 percent for joints made with bolts $1^1/_2$-inch in diameter, and proportionally for intermediate diameters. No increase in the tabulated values is allowed for perpendicular-to-grain loading (Q value).

Clearance requirements for end, edge, and bolt spacing distance shall be as shown below. All distances are measured from the end or side of the wood member to the center of the bolt hole. For members which are subject to load reversals the larger controlling distances shall be used for design. For parallel-to-grain loading, the minimum distances for full design load:

1. In tension, minimum end distance shall be 7 times the bolt diameter;
2. In compression, minimum end distance shall be 4 times the bolt diameter; and
3. In tension or compression, the minimum edge distance shall be 1.5 times the bolt diameter.

For perpendicular-to-grain loading, the minimum distance for full design load:

1. Minimum end distance shall be 4 times the bolt diameter;
2. Edge distance toward which the load is acting shall be at least 4 times the bolt diameter; and
3. Distance on the opposite edge shall be at least 1.5 bolt diameters.

Minimum clearance (spacing) between adjacent bolts in a row shall be 4 times the bolt diameter, measured center-to-center of the bolt holes.

When more than two bolts are used in a line parallel to the axis of the side member, additional requirements shall be followed as shown in the AITC manual.

Lag Screw Connections

Design values for lag screws subject to withdrawal loading are found in AITC Table 6.27. Values for wood having a specific gravity of 0.51 for Douglas Fir-Larch or 0.42 for Hem-Fir shall be assumed when using the table. The withdrawal values are in pounds per inch of penetration of the threaded part of the lag screw into the side grain of the member holding the point, with the axis of the screw perpendicular to that member. The maximum load on a given screw shall not exceed the allowable tensile strength of the screw at the root section.

AITC recommends against subjecting lag screws to end-grain withdrawal loading. However, if this condition cannot be avoided, the design value shall be 75 percent of the corresponding value for withdrawal from the side grain.

Values in the Group II wood species column shall be used for Douglas Fir-Larch and the Group III wood species column shall be used for Hem-Fir. When the load is applied at an angle to the grain, as is the case with falsework bracing, the design value shall be obtained from the Hankinson formula shown in the AITC manual.

When lag screws are subjected to a combined lateral and withdrawal loading, as would be the case with longitudinal bracing when the falsework bents are skewed, the effect of the lateral and withdrawal forces shall be determined separately. The withdrawal component of the applied load shall not exceed the allowable value in withdrawal. The lateral component of the applied load shall not exceed the allowable lateral load value.

Lag screws shall be inserted in lead holes as follows:

1. The clearance hole for the shank shall have the same diameter as the shank, and the same depth of penetration as the length of unthreaded shank;
2. The lead hole for the threaded portion shall have a diameter equal to 60 to 75 percent of the shank diameter and a length equal to at least the length of the threaded portion. The larger percentile figure in each range shall apply to screws of the greater diameters used in Group II wood species;
3. The threaded portion of the screw shall be inserted in its lead hole by turning with a wrench, not by driving with a hammer; and
4. To facilitate insertion, soap or other lubricant shall be used on the screws or in the lead hole.

Drift Pin and Drift Bolt Connections

When drift pins or drift bolts are used, the required length and penetration shall be determined using the following criteria. The lateral load-carrying capacity of drift pins and drift bolts driven into the side grain of a wood member shall be limited to 75 percent of the design values for a common bolt of the same diameter and length in the main member. For drift pin connections, the pin penetration into the connected members shall be increased to compensate for the absence of a bolt head and nut. For drift bolts or pins driven into the end grain of a member, the lateral load-carrying capacity shall be limited to 60 percent of the allowable side grain load (perpendicular to grain value) for an equal diameter bolt with nut. To develop this allowable load the drift bolt or pin shall penetrate at least 12 diameters into the end grain. To fully develop the allowable load of the drift bolts or pins, they shall be driven into predrilled holes, $^1/_{16}$-inch less in diameter than the drift pin or bolt diameter.

The criteria shown in the AITC *Timber Construction Manual-Third Edition* shall apply to drift bolt or pin connection allowable loads for the following conditions:

1. Withdrawal resistance; and
2. When there are more than two drift bolts or pins in a joint, allowable loads shall be further reduced by applying applicable modification factors shown in the AITC Table 6.3.

Nailed and Spiked Joints

Joints using nails or spikes shall conform to the provisions of AITC. For side grain withdrawal, the values in AITC Table 6.35 for wood having a specific gravity of 0.51 for Douglas Fir-Larch and a specific gravity of 0.42 for Hem-Fir shall be used. End grain withdrawal shall not be used. For lateral loading, the values in AITC Table 6.36 for wood species Group II for Douglas Fir-Larch and wood species Group III for Hem-Fir shall be used. Diameters listed in the tables apply to fasteners before application of any protective coating.

When more than one nail or spike is used in a joint, the total design value for the joint in withdrawal or lateral resistance shall be the sum of the design values for the individual nails or spikes.

The tabulated design values for lateral loads are valid only when the nail penetrates into the main member at least 11 diameters for Douglas Fir-Larch and 13 diameters for Hem-Fir. Note that the values are maximum values for the type and size of fastener shown. The tabulated values shall not be increased even if the actual penetration is exceeded.

When main member penetration is less than 11 diameters for Douglas Fir-Larch and 13 diameters for Hem-Fir, the design value shall be determined by straight-line interpolation between zero and the tabulated load, except that penetration shall not be less than $^1/_3$ of that specified.

Double-headed or duplex nails used in falsework and formwork construction are shorter than common wire nails or box nails of the same size designation. They have less penetration into the main member and therefore their load-carrying capacity shall be adjusted accordingly.

Nail and spike minimum spacing in timber connections shall be as follows:

1. The average center-to-center distance between adjacent nails, measured in any direction, shall not be less than the required penetration into the main member for the size of nail being used; and

2. The minimum end distance in the side member, and the minimum edge distance in both the side member and the main member, shall not be less than $^1/_2$ of the required penetration.

Allowable values for withdrawal and lateral load resistance are reduced when toe nails are used in accordance with the following:

1. For withdrawal loading, the design load shall not exceed $^2/_3$ of the value shown in the applicable design table; and
2. For lateral loading, the design load shall not exceed $^5/_6$ of the value shown in the applicable design table.

Toe nails are recommended to be driven at an approximate angle of 30 degrees with the piece and started approximately $^1/_3$ of the length of the nail from the end or side of the piece.

Timber Connection Adjustment for Duration of Load

Tabulated values for timber fasteners are for normal duration of load and may be increased for short duration loading, except for connections used in falsework and formwork for post tensioned structures and staged construction sequences. Duration of load adjustment for timber connections shall not be allowed for all post tensioned structures and for staged construction sequences where delayed and/or staged loading occurs for any type of concrete structure. The adjustment for duration of load as described in this section applies only to design values for timber connectors, such as nails, bolts, and lag screws. Allowable stresses for timber and structural steel components used in the connection, as described in Section 6-02.3(17)B, are maximums and thus shall not be increased.

Tabulated values for nails, bolts, and lag screws may be adjusted by the following duration-of-load factors:

1. 1.25 for falsework design governed by the minimum design horizontal load or greater (three percent or greater of the dead load);
2. 1.33 for falsework design governed by wind load; and
3. 2.00 for falsework design governed by impact loading.

6-02.3(17)J Face Lumber, Studs, Wales, and Metal Forms

Elements of this section shall be designed for the loads, allowable stresses, deflections, and conditions which pertain from other subsections of Section 6-02.3(17).

Forms battered or inclined above the concrete will tend to lift up as concrete is placed and shall have positive anchorage or counterweights designed to resist uplift and shall be shown in the formwork plans. Where the concrete pouring sequence causes fresh concrete to be significantly higher along one side of tied forms than the opposite side, a positive form anchorage system shall be designed capable of resisting the imbalance of horizontal thrust, and prevent the dislocation and sliding of the entire form unit.

Wooden forms shall be faced with smooth sanded, exterior plywood. This plywood shall meet the requirements of the National Bureau of Standards, U.S. Product Standard PS 1, and the Design Specification of the American Plywood Association (APA). Each full sheet shall bear the APA stamp. The Contractor shall list in the form plans the grade and class of plywood. If the Engineer approves the manufacturer’s certification of structural properties, the Contractor may use plywood that does not carry the APA stamp. Plywood panels stamped “shop” or “shop cutting,” shall not be used.

Plyform is an APA plywood specifically designed and manufactured for concrete forming. Plyform differs from conventional exterior plywood grades in strength and the exterior face panels are sanded smooth and factory oiled. Likewise, there is a significant difference between grades designated Class 1, Class 2, and Structural I Plyform.

The grades of plywood for various form applications shall be as follows:

1. **Traffic and Pedestrian Barriers** (except those that will receive an architectural surface treatment) — Plywood used for these surfaces shall be APA grade High Density Overlaid (HDO) Plyform Class I. But if the Contractor coats the form to prevent it from leaving joint and grain marks on the surface, plywood that meets or exceeds APA grades B-B Plyform Class I or B-C (Group I species) may be used. Under this option, the Contractor shall provide for the Engineer's approval a 4-foot square, test panel of concrete formed with the same plywood and coating as proposed in the form plans. This panel shall include one form joint along its centerline. The Contractor shall apply coating material, according to the manufacturer's instructions, before applying chemical release agents.
2. **Other Exposed Surfaces** (all but those on traffic and pedestrian barriers) — Plywood used to form these surfaces shall meet or exceed the requirements of APA grades B-B Plyform Class I or B-C (Group I series). If one face is less than B quality, the B (or better) face shall contact the concrete.
3. **Unexposed Surfaces** (such as the undersides of roadway slabs between girders, the interiors of box girders, etc., and traffic and pedestrian barriers where surfaces will receive an architectural treatment) — Plywood used to form these surfaces may be APA grade CDX, provided the Contractor complies with stress and deflection requirements stated elsewhere in these Specifications.

Form joints on an exposed surface shall be in a horizontal or vertical plane. But in wingwalls and box girders, side form joints shall be placed at right angles and parallel to the roadway grade. Joints parallel to studs or joists shall be backed by a stud or joist. Joints at right angles to studs and joists shall be backed by a stud or other backing the Engineer approves. Perpendicular backing is not required if studs or joists are spaced:

1. Nine inches or less on center and covered with $^1/_2$-inch plywood, or
2. Twelve inches or less on center and covered with $^3/_4$-inch plywood.

The face grain of plywood shall run perpendicular to studs or joists unless shown otherwise on the Contractor's formwork plans and approved by the Engineer. Proposals to deviate from the perpendicular orientation shall be accompanied by supporting calculations of the stresses and deflections.

Forming for all exposed curved surfaces shall follow the shape of the curve shown in the Contract Plans and shall not be chorded except as follows. On any retaining wall that follows a horizontal circular curve, the wall stems may be a series of short chords if:

1. The chords within the panel are the same length, unless otherwise approved by the Engineer;
2. The chords do not vary from a true curve by more than $^1/_2$-inch at any point; and
3. All panel points are on the true curve.

Where architectural treatment is required, the angle point for chords in wall stems shall fall at vertical rustication joints.

For exposed surfaces of abutments, wingwalls, piers, retaining walls, and columns, the Contractor shall build forms of plywood at least $^3/_4$-inch thick with studs no more than 12-inches on center. The Engineer may approve exceptions, but deflection of the plywood, studs, or wales shall never exceed $^1/_{360}$ of the span (or $^1/_{270}$ of the span for unexposed surfaces, including the bottom of the deck slab between girders).

All form plywood shall be at least $^1/_2$-inch thick except on sharply curved surfaces. There, the Contractor may use $^1/_4$-inch plywood if it is backed firmly with heavier material.

Round columns or rounded pier shafts shall be formed with a self-supporting metal shell form or form tube that leaves a smooth, nonspiralling surface. Wood forms are not permitted.

Metal forms shall not be used elsewhere unless the Engineer is satisfied with the surface and approves in writing. The Engineer may withdraw approval for metal forms at any time. If permitted to use a combination of wood and metal in forms, the Contractor shall coat the forms so that the texture produced by the wood matches that of the metal. Aluminum shall not be used for metal forms.

For design purposes, the Contractor shall assume that on vertical surfaces concrete exerts 150 pounds per sq. ft. per foot of depth. However, when the depth is reached where the rate of placement controls the pressure, the following table applies:

Rate of Placing Feet per Hour	**Pressure, Pounds per Square Foot for Temperature of Concrete as Shown**	
	60ºF	**70ºF and above**
2	470	375
3	640	565
4	725	625
5	815	690
6	900	750
7	990	815
8	1,075	875
9	1,165	935
10	1,250	1,000
15	1,670	1,300

The pressures in the above table have been increased to provide an allowance for the vibration and impact.

All corners shall be beveled $^3/_4$-inch. However, footings, footing pedestals, and seals need not be beveled unless required in the Plans.

All forms shall be as mortar-tight as possible with no water standing in them as the concrete is placed.

The Contractor shall apply a parting compound on forms for exposed concrete surfaces. This compound shall be a chemical release agent that permits the forms to separate cleanly from the concrete. The compound shall not penetrate or stain the surface and shall not attract dirt or other foreign matter. After the forms are removed, the concrete surface shall be dust-free and have a uniform appearance. The Contractor shall apply the compound at the manufacturer's recommended rate to produce a surface free of dusting action and yet provide easy removal of the forms.

If an exposed concrete surface will be sealed, the release agent shall not contain silicone resin. Before applying the agent, the Contractor shall provide the Engineer a written statement from the manufacturer stating whether the resin in the base material is silicone or nonsilicone.

The Contractor shall select a parting compound from the current Qualified Products List, or submit to the Engineer a sample of the parting compound at least ten working days before its use. Approval or disapproval shall be based on laboratory test results or selection off the current Qualified Products List.

The Engineer may reject any forms that will not produce a satisfactory surface.

6-02.3(17)K Concrete Forms on Steel Spans

Concrete forms on all steel structures shall be removable and shall not remain in place. Where needed, the forms shall have openings for truss or girder members. Each opening shall be large enough to leave at least 1 1/2-inches between the concrete and steel on all sides of the steel member after the forms have been removed. Unit contract prices cover all costs related to these openings.

Any form support for a roadway slab that rests on a plate girder flange shall apply the load within 6-inches of the girder web centerline. The Contractor shall not weld any part of the form to any steel member.

The compression member or bottom connection of cantilever formwork support brackets shall bear either within six inches maximum vertically of the bottom flange or within six inches maximum horizontally of a vertical web stiffener. The Contractor shall also furnish and install temporary struts and ties to prevent rotation of the steel girder. Partial depth cantilever formwork support brackets that do not conform to the above requirements shall not be used, unless the Contractor submits details showing the additional formwork struts and ties used to brace the steel girder against web distortion caused by the partial depth bracket, and receives the Engineer's approval of the submittal.

If the Engineer permits bolt holes in the web to support form brackets, the holes shall be shop drilled unless otherwise approved by the Engineer. The Contractor shall fill the holes with fully torqued AASHTO M 164 bolts per Section 6-03.3(33). Each bolt head shall be placed on the exterior side of the web. There shall be no holes made in the flanges.

6-02.3(17)L Finishing Machine Support System

Before using any finishing machine, the Contractor shall obtain the Engineer's approval of detailed drawings that show the system proposed to support it. The Contractor shall not attach this (or any other) equipment support system to the sides or suspend it from any girder unless the Engineer permits. The Engineer will not permit such a method if it will unduly alter stress patterns or create too much stress in the girder.

6-02.3(17)M Restricted Overhead Clearance Sign

The Contractor shall notify the Engineer not less than 15 working days before the anticipated start of each falsework and girder erection operation whenever such falsework or girders will reduce clearances available to the public traffic. Falsework openings shall not be more restrictive to traffic than shown in the Contract Plans.

Where the height of vehicular openings through falsework is less than 15-feet, a W 12-2 "Low Clearance Symbol Sign" shall be erected on the shoulder in advance of the falsework and two or more W 12-301 and/or W 12-302 signs shall be attached to the falsework to provide accurate usable clearance information over the entire falsework opening. The posted low clearance shall include an allowance for anticipated falsework girder deflection (rounded-up to the next whole inch) due to design dead load, including all successive concrete pours. W 12-302 signs shall be used to designate prominent clearance restrictions and limits of usable clearance. In addition, where the clearance is less than the legal height limit (14-feet 0-inches), a W 12-2 sign shall be erected in advance of the nearest intersecting road or wide point in the road at which a vehicle can detour or turn around. A W 13-501 sign indicating the distance to the low clearance shall be installed below the advance sign. The Engineer will furnish the above noted signs and the Contractor shall erect and maintain them, all in accordance with Section 1-10.3(3).

When erecting falsework that restricts overhead clearance above a railroad track, the Contractor shall immediately (as soon as the restriction occurs) place restricted overhead clearance signs. Sign details are shown in the Standard Plans. Unit contract prices cover all costs relating to these signs.

6-02.3(17)N Removal of Falsework and Forms

The Engineer will decide, on the basis of post-placement curing conditions, the exact number of days that shall elapse before form or falsework removal. If the Engineer does not specify otherwise, the Contractor may request to remove forms based on the criteria listed in the table below. Both compressive strength and number of days criteria must be met if both are listed. The number of days shall be from the time of the last pour the forms support. In no case shall the Contractor remove forms or falsework without the Engineer's approval.

Concrete Placed In	Percent of Specified Minimum Compressive Strength	Number of Days
Columns, wall faces, mass piers and abutments (except pier caps), traffic and pedestrian barriers, and any other side form not supporting the concrete weight.[1]	—	3
Pier caps continuously supported.[2]	60	3
Sidewalks not supported on bridge roadway slabs.[2]	70	—
Crossbeams, caps, pier caps not continuously supported, struts and top slabs on concrete box culverts, inclined columns and inclined walls.[2,3]	80	5
Roadway slabs supported on wood or steel stringers or on steel or prestressed concrete girders.[2]	80	10
Box girders, T-beam girders, and flat-slab superstructure.[2,3]	80	14
Arches.[2,3]	—	21

[1]Where forms do not support the load of concrete.

[2]Where forms support the load of concrete.

[3]Where continuous spans are involved, the time for all spans will be determined by the last concrete placed affecting any span.

Before releasing supports from beneath beams and girders, the Contractor shall remove forms from columns to enable the Engineer to inspect the column concrete.

The Contractor may remove the side forms of footings 24 hours after concrete placement if a curing compound is applied immediately. But this compound shall not be applied to that area of the construction joint between the footing and the column or wall.

The Contractor may remove side forms, traffic barrier forms, and pedestrian barrier forms after 24 hours if these forms are made of steel or dense plywood, an approved water reducing additive is used, and the concrete reaches a compressive strength of 1400 psi before form removal. This strength shall be proved by test cylinders made from the last concrete placed into the form. The cylinders shall be cured according to WSDOT FOP for AASHTO T 23.

Wet curing shall comply with the requirements of Section 6-02.3(11). The concrete surface shall not become dry during form removal or during the entire curing period.

Before placing forms for traffic and pedestrian barriers, the Contractor shall completely release all falsework under spans.

Before releasing forms under concrete subjected to temperatures colder than 50ºF, the Contractor shall first prove that the concrete meets desired strength — regardless of the time that has elapsed.

The Engineer may approve leaving in place forms for footings in cofferdams or cribs. This decision will be based on whether removing them would harm the cofferdam or crib and whether the forms will show in the finished structure.

All cells of a box girder structure which have permanent access shall have all forms completely removed, including the roadway deck forms. All debris and all projections into the cells shall be removed. Unless otherwise shown in the Plans, the roadway slab interior forms in all other cells where no permanent access is available, may be left in place.

Falsework and forms supporting sloping exterior webs shall not be released until the roadway deck and deck overhang concrete has obtained its removal strength and number of days criteria listed in the table above. Stem reshoring shall not be used.

Open joints shown in the Plans shall have all forms completely removed, including Styrofoam products and form anchors, allowing the completed structure to move freely.

If the Contractor intends to support or suspend falsework and formwork from the bridge structure while the falsework and formwork is being removed, the Contractor shall submit a falsework and formwork removal plan and calculations for review and approval. The falsework and formwork removal plan shall include the following:

1. The location and size of any cast-in-place falsework lowering holes and how the holes are to be filled;
2. The location, capacity, and size of any attachments, beams, cables, and other hardware used to attach to the structure or support the falsework and formwork;
3. The type, capacity and factor of safety, weight, and spacing of points of reaction of lowering equipment; and
4. The weight at each support point of the falsework and formwork being lowered.

All other forms shall be removed whether above or below the level of the ground or water. Sections 6-02.3(7) and 6-02.3(8) govern form removal for concrete exposed to sea water or to alkaline water or soil. The forms inside of hollow piers, girders, abutments, etc. shall be removed through openings shown in the Plans or approved by the Engineer.

6-02.3(17)O Early Concrete Test Cylinder Breaks

The fabrication, curing, and testing of the early cylinders shall be the responsibility of the Contractor. Early cylinders are defined as all cylinders tested in advance of the design age of 28 days whose purpose is to determine the in-place strength of concrete in a structure prior to applying loads or stresses. The Contractor shall retain a testing laboratory to perform this work. Testing laboratories' equipment shall be calibrated within one year prior to testing and testers shall be either ACI certified or qualified in accordance with AASHTO R 18.

The concrete cylinders shall be molded in accordance with WSDOT FOP for AASHTO T 23 from concrete last placed in the forms and representative of the quality of concrete placed in that pour.

The cylinders shall be cured in accordance with WSDOT FOP for AASHTO T 23. The Engineer may approve the use of cure boxes meeting the requirements of this test method. Special cure boxes to enhance cylinder strength will not be allowed.

The concrete cylinders shall be tested for compressive strength in accordance with AASHTO T 22. The number of early cylinder breaks shall be in accordance with the Contractor's need and as approved by the Engineer.

The Contractor shall furnish the Engineer with all test results, proof of equipment calibration, and tester's certification. The test results will be reviewed and approved before any forms are removed. The Contractor shall not remove forms without the approval of the Engineer.

All costs in connection with furnishing cylinder molds, fabrication, curing, and testing of early cylinders shall be included in the unit contract prices for the various bid items of work involved.

6-02.3(18) Placing Anchor Bolts

The Contractor shall comply with the following requirements in setting anchor bolts in piers, abutments, or pedestals:

1. If set in the wet concrete, the bolts shall be accurately placed before the concrete is placed.
2. If the bolts are set in drilled holes, hole diameter shall exceed bolt diameter by at least 1-inch. Grouting shall comply with Section 6-02.3(20).
3. If the bolts are set in pipe, grouting shall comply with Section 6-02.3(20).
4. If freezing weather occurs before bolts can be grouted into sleeves or holes, they shall be filled with an approved antifreeze solution (non-evaporating).

6-02.3(19) Bridge Bearings

6-02.3(19)A Vacant

6-02.3(19)B Bridge Bearing Assemblies

For all fixed, sliding, or rolling bearings, the Contractor shall:

1. Machine all sliding and rolling surfaces true, smooth, and parallel to the movement of the bearing;
2. Polish all sliding surfaces;
3. Anchor expansion bearings securely, setting them true to line and grade;

4. Avoid placing concrete in such a way that it might interfere with the free action of any sliding or rolling surface.

Grout placement under steel bearings shall comply with Section 6-02.3(20).

6-02.3(20) Grout for Anchor Bolts and Bridge Bearings

Grout shall be a prepackaged grout, mixed, placed, cured as recommended by the manufacturer, or the grout shall be produced using Type I or II Portland cement, fine aggregate Class 1 or Class 2, and water, in accordance with these Specifications.

Grout shall meet the following requirements:

Requirement	Compressive Strength
Test Method	AASHTO Test Method T 106
Values	4,000 psi @ 7 days

Grout shall be a workable mix with flowability suitable for the intended application.

If the Contractor elects to use a prepackaged grout, a material sample and laboratory test data from an independent testing laboratory shall be submitted to the Engineer for approval with the request for approval of material sources.

If the Contractor elects to use a grout consisting of Type I or II Portland cement, fine aggregate Class 1, admixture, and water, the mix proportions and laboratory test data from an independent test laboratory shall be submitted to the Engineer for approval with the request for approval of material sources.

The Contractor shall receive approval from the Engineer before using the grout.

Field grout cubes shall be made in accordance with WSDOT Test Method 813 for either prepackaged grout or a contractor provided mix when requested by the Engineer, but not less than one per bridge pier or one per day.

Before placing grout, the concrete on which it is to be placed shall be thoroughly cleaned, roughened, and wetted with water to ensure proper bonding. The grout pad shall be cured as recommended by the manufacturer or kept continuously wet with water for three days. The grout pad may be loaded when a minimum of 4000 psi compressive strength is attained.

Before placing grout into anchor bolt sleeves or holes, the cavity shall be thoroughly cleaned and wetted to ensure proper bonding.

To grout bridge bearing masonry plates, the Contractor shall:

1. Build a form approximately 4-inches high with sides 4-inches outside the base of each masonry plate;
2. Fill each form to the top with grout;
3. Work grout under all parts of each masonry plate;
4. Remove each form after the grout has hardened;
5. Remove the grout outside each masonry plate to the base of the masonry plate;
6. Bevel off the grout neatly to the top of the masonry; and
7. Place no additional load on the masonry plate until the grout has set at least 72 hours.

After all grout under the masonry plate and in the anchor bolt cavities has attained a minimum strength of 4,000 psi, the anchor bolt nuts shall be tightened to snug-tight. "Snug-tight" means either the tightness reached by (1) a few blows from an impact wrench, or (2) the full effort of a person using a spud wrench. Once the nut is snug-tight, the anchor bolt threads shall be burred just enough to prevent loosening of the nut.

6-02.3(21) Drainage of Box Girder Cells

To drain box girder cells, the Contractor shall provide and install, according to details in the Plans, short lengths of nonmetallic pipe in the bottom slab at the low point of each cell. The pipe shall have a minimum inside diameter of 4-inches. If the difference in plan elevation is 2-inches or less, the Contractor shall install pipe in each end of the box girder cell. All drainage holes shall be screened in accordance with the Plan details.

6-02.3(22) Drainage of Substructure

The Contractor shall use weep holes and gravel backfill that complies with Section 9-03.12(2) to drain fill material behind retaining walls, abutments, tunnels, and wingwalls. To maintain thorough drainage, weep holes shall be placed as low as possible. Weep holes shall be covered with geotextile meeting the requirements of Section 9-33.2, Table 2 Class C before backfilling. Geotextile screening shall be bonded to the concrete with an approved adhesive. Gravel backfill shall be placed and compacted as required in Section 2-09.3(1)E. In addition, if the Plans require, tiling, French or rock drains, or other drainage devices shall be installed.

If underdrains are not installed behind the wall or abutment, all backfill within 18-inches of weep holes shall comply with Section 9-03.12(4). Unless the Plans require otherwise, all other backfill behind the wall or abutment shall be gravel backfill for walls.

6-02.3(23) Opening to Traffic

Bridges with a roadway slab made of Portland cement concrete shall remain closed to all traffic, including construction equipment, until the concrete has reached the 28-day specified compressive strength. This strength shall be determined with cylinders made of the same concrete as the roadway and cured under the same conditions. A concrete deck bridge shall never be opened to traffic earlier than ten days after the deck concrete was placed and never before the Engineer has approved.

For load restrictions on bridges under construction, refer to Section 6-01.6.

6-02.3(24) Reinforcement

Although the Plans normally include a bar list and bending diagram, these shall be used at the Contractor's risk. The Engineer advises the Contractor to check the order from the Plans.

Before delivery of the reinforcing bars, the Contractor shall submit to the Engineer two informational copies of the supplemental bending diagrams.

Various steel reinforcing bars, including those in crossbeams, may be shown as straight in the bar list sheets of the Plans. The Contractor shall bend these bars as required to conform to the configuration of the structure and as detailed in the Plans.

6-02.3(24)A Field Bending

If the Plans call for field bending of steel reinforcing bars, the Contractor shall bend them in keeping with the structure configuration and the Plans and Specifications.

Bending steel reinforcing bars partly embedded in concrete shall be done as follows:

Field bending shall not be done:

1. On bars size No. 14 or No. 18,
2. When air temperature is lower than 45ºF,
3. By means of hammer blows or pipe sleeves, or
4. While bar temperature is in the range of 400º to 700ºF.

In field-bending steel reinforcing bars, the Contractor shall:

1. Make the bend gradually;
2. Apply heat as described in Tables 2 and 3 for bending bar sizes No. 6 thru No. 11 and for bending bar sizes No. 5 and smaller when the bars have been previously bent. Previously unbent bars of sizes No. 5 and smaller may be bent without heating;
3. Use a bending tool equipped with a bending diameter as listed in Table 1;
4. Limit any bend to these maximums — 135 degrees for bars smaller than size No. 9, and 90 degrees for bars size No. 9 and No. 11;
5. Straighten by moving a hickey bar (if used) progressively around the bend.

In applying heat for field-bending steel reinforcing bars, the Contractor shall:

1. Use a method that will avoid damages to the concrete;
2. Insulate any concrete within 6-inches of the heated bar area;
3. Ensure, by using temperature-indicating crayons or other suitable means, that steel temperature never exceeds the maximum temperatures shown in Table 2 below;
4. Maintain the steel temperature within the required range shown in Table 2 below during the entire bending process;
5. Apply two heat tips simultaneously at opposite sides of bars larger than size No. 6 to assure a uniform temperature throughout the thickness of the bar. For size No. 6 and smaller bars, apply two heat tips, if necessary;
6. Apply the heat for a long enough time that within the bend area the entire thickness of the bar — including its center — reaches the required temperature;
7. Bend immediately after the required temperature has been reached;
8. Heat at least as much of the bar as Table 3 below requires;
9. Locate the heated section of the bar to include the entire bending length; and
10. Never cool bars artificially with water, forced air, or other means.

Table 1
Bending Diameters for Field-Bending Reinforcing Bars

	Bend Diameter/Bar Diameter Ratio	
Bar Size	**Heat Not Applied**	**Heat Applied**
No. 4, No. 5	8	8
No. 6 through No. 9	Not Permitted	8
No. 10, No. 11	Not Permitted	10

The minimum bending diameters for stirrups and ties for No. 4 and No. 5 bars when heat is not applied shall be specified in Section 9-07.

Table 2
Preheating Temperatures for Field-Bending Reinforcing Bars

	Temperature (F)	
Bar Size	**Minimum**	**Maximum**
No. 4	1,200	1,250
No. 5, No. 6	1,350	1,400
No. 7 through No. 9	1,400	1,450
No. 10, No. 11	1,450	1,500

Table 3
Minimum Bar Length to be Heated (d = nominal diameter of bar)

	Bend Angle		
Bar Size	**45°**	**90°**	**135°**
No. 4 through No. 8	8d	12d	15d
No. 9	8d	12d	Not Permitted
No. 10, No. 11	9d	14d	Not Permitted

6-02.3(24)B Protection of Materials

The Contractor shall protect reinforcing steel from all damage. When placed into the structure, the steel shall be free from dirt, loose rust or mill scale, paint, oil, and other foreign matter.

When transporting, storing, or constructing in close proximity to bodies of salt water, plain and epoxy-coated steel reinforcing bar shall be kept in enclosures that provide protection from the elements.

If plain or epoxy-coated steel reinforcing bar is exposed to mist, spray, or fog that may contain salt, it shall be flushed with fresh water prior to concrete placement.

When the Engineer requires protection for reinforcing steel that will remain exposed for a length of time, the Contractor shall protect the reinforcing steel:

1. By cleaning and applying a coat of paint Formula No. A-9-73 over all exposed surfaces of steel, or
2. By cleaning and painting paint Formula No. A-9-73 on the first 6-inches of the steel bars protruding from the concrete and covering the bars with polyethylene sleeves.

The paint shall have a minimum dry film thickness of 1 mil.

6-02.3(24)C Placing and Fastening

The Contractor shall position reinforcing steel as the Plans require and shall ensure that the steel does not move as the concrete is placed.

When spacing between bars is 1 foot or more, they shall be tied at all intersections. When spacing is less than 1 foot, every other intersection shall be tied. If the Plans require bundled bars, they shall be tied together with wires at least every 6-feet. All epoxy-coated bars in the top mat of the roadway slab shall be tied at all intersections. Other epoxy-coated bars shall also be tied at all intersections, but shall be tied at alternate intersections when spacing is less than 1 foot in each direction. Wire used for tying epoxy-coated reinforcing steel shall be plastic coated. **Tack welding is not permitted on reinforcing steel**.

Abrupt bends in the steel are permitted only when one steel member bends around another. Vertical stirrups shall pass around main reinforcement or be firmly attached to it.

For slip-formed concrete, the reinforcing steel bars shall be tied at all intersections and cross braced to keep the cage from moving during concrete placement. Cross bracing shall be with additional reinforcing steel. Cross bracing shall be placed both longitudinally and transversely.

After reinforcing steel bars are placed in a traffic or pedestrian barrier and prior to slip-form concrete placement, the Contractor shall check clearances and reinforcing steel bar placement. This check shall be accomplished by using a template or by operating the slip-form machine over the entire length of the traffic or pedestrian barrier. All clearance and reinforcing steel bar placement deficiencies shall be corrected by the Contractor before slip-form concrete placement.

Mortar blocks (or other approved devices) shall be used to maintain the concrete coverage required by the Plans. The Mortar blocks shall:

1. Have a bearing surface measuring not greater than 2-inches in either dimension, and
2. Have a compressive strength equal to that of the concrete in which they are embedded.

In slabs, each mortar cube shall have either: (1) a grooved top that will hold the reinforcing bar in place, or (2) an embedded wire that protrudes and is tied to the reinforcing steel. If this wire is used around epoxy-coated bars, it shall be coated with plastic.

Mortar blocks may be accepted on a Manufacturers Certificate of Compliance, which shall include test results on sets of two 2-inch square specimens per AASHTO T 106. Each pair of specimens shall represent 2,500 or fewer mortar blocks and shall be made of the same mortar as the blocks and cured under the same conditions.

In lieu of mortar blocks, the Contractor may use metal or plastic chair supports to hold uncoated bars. Any surface of a metal chair support that will not be covered by at least $^1/_2$-inch of concrete shall be one of the following:

1. Hot-dip galvanized after fabrication in keeping with AASHTO M 232 Class D,
2. Coated with plastic firmly bonded to the metal. This plastic shall be at least $^3/_{32}$-inch thick where it touches the form and shall not react chemically with the concrete when tested in the State Materials Laboratory. The plastic shall not shatter or crack at or above -5ºF and shall not deform enough to expose the metal at or below 200ºF, or
3. Stainless steel that meet the requirements of ASTM A 493, Type 302. Stainless steel chair supports are not required to be galvanized or plastic coated.

In lieu of mortar blocks, epoxy-coated reinforcing bars may be supported by one of the following:

1. Metal chair supports coated entirely with a dielectric material such as epoxy or plastic,
2. Other epoxy-coated reinforcing bars, or
3. Plastic chair supports.

Plastic chair supports shall be lightweight, non-porous, and chemically inert in concrete. Plastic chair supports shall have rounded seatings, shall not deform under load during normal temperatures, and shall not shatter or crack under impact loading in cold weather. Plastic chair supports shall be placed at spacings greater than 1 foot along the bar and shall have at least 25 percent of their gross place area perforated to compensate for the difference in the coefficient of thermal expansion between plastic and concrete. The shape and configuration of plastic supports shall permit complete concrete consolidation in and around the support.

In roadway and sidewalk slabs, the Contractor shall place reinforcing steel mats carefully to provide the required concrete cover. A "mat" is two layers of steel. Top and bottom mats shall be supported enough to hold both in their proper positions. If No. 4 bars make up the lower layer of steel in a mat, it shall be blocked at not more than 3-foot intervals (or 4-foot intervals for bars No. 5 and larger). Wire ties to girder stirrups shall not be considered as blocking. To provide a rigid mat, the Contractor shall add other supports and tie wires to the top mat as needed.

If a bar will interfere with a bridge drain, it shall be bent in the field to bypass the drain.

Clearances shall be at least:

4-inches between:	Main bars and the top of any concrete masonry exposed to the action of salt or alkaline water.
2$^1/_2$-inches between:	Adjacent bars in a layer. Slab bars and the top of the roadway slab. Main bars and the surface of concrete deposited against earth (without intervening forms).
2-inches between:	Adjacent layers. Main bars and the surface of concrete (except in walls and slabs). Reinforcing bars and the faces of forms for exposed aggregate finish.
1$^1/_2$-inches between:	Main bars and the surface of concrete in retaining walls. Slab bars and the top of the slab (except roadway slabs). Stirrups and ties and the surface of the concrete.
1-inch between:	Slab bars and the bottom of the slab. Curb or sidewalk bars and the surface of the concrete.

Reinforcing steel bars shall not vary more than the following tolerances from their position shown in the Plans:

Members 10-inches or less in thickness	±1/4 in.
Members more than 10-inches in thickness	±3/8 in.
Drilled Shafts top of rebar cage elevation	+6 in./-3 in.
Except:	
The distance between the nearest reinforcingsteel bar surface and the top surface of the roadway deck slab	+1/4 in.
Longitudinal spacing of bends and ends of bars	±1 in.
Length of bar laps	-1 1/2 in.
Embedded length	
No 3 through No. 11	-1 in.
No. 14 through No. 18	-2 in.
When reinforcing steel bars are to be placedat equal spacing within a plane:	
Stirrups and ties	±1 in.
All other reinforcement	±1 bar dia.

Before placing any concrete, the Contractor shall:

1. Clean all mortar from reinforcement, and
2. Obtain the Engineer's permission to place concrete after the Engineer has inspected the placement of the reinforcing steel. (Any concrete placed without the Engineer's permission shall be rejected and removed.)

6-02.3(24)D Splicing

The Contractor shall supply steel reinforcing bars in the full lengths the Plans require. Unless the Engineer approves in writing, the Contractor shall not change the number, type, or location of splices.

The Engineer may permit the Contractor to use thermal or mechanical splices in place of the method shown in the Plans if they are of an approved design. Use of a new design may be granted if:

1. The Contractor provides technical data and proof from the manufacturer that the design will perform satisfactorily, and
2. Sample splices and materials from the manufacturer pass the Engineer's tests.

After a design has been approved, any changes in detail or material shall require new approval.

The Contractor shall:

1. Not lap-splice reinforcing bars Nos. 14 or 18.
2. Not permit any welded or mechanical splice to deviate in alignment more than 1/4-inch per 3 1/2-feet of bar.
3. Distribute splices evenly, grouping them together only at points of low tensile stress.
4. Ensure at least 2-inches clearance between any splice and the nearest bar or the surface of the concrete (or 1 1/2-inch for the length of the sleeve on mechanical splices).

5. Rigidly clamp or wire all splices in a way the Engineer approves.
6. Place lap-spliced bars in contact for the length of the splice and tie them together near each end.
7. Securely fasten the ends and edges of welded-wire-fabric reinforcement, overlapping them enough to maintain even strength.

6-02.3(24)E Welding Reinforcing Steel

Welding of steel reinforcing bars shall conform to the requirements of the Special Provisions, Plans, and these Specifications.

When welding is required, steel reinforcing bars shall be supplied that are suitable for welding. Steel which is to be welded shall have a maximum carbon equivalent of 0.65 percent. The carbon equivalent shall be determined by the following formula:

$$CE = \% C + \% Mn/6 + \% Cu/40 + \% Ni/20 + \% Cr/10 - \% Mo/50 - \% V/10$$

In addition, carbon shall not exceed 0.45 percent nor manganese 1.30 percent.

Before any welding begins, the Contractor shall obtain the Engineer's approval of a written welding procedure for each type of welded splice to be used, including the procedure specifications and joint details. The procedure specifications shall specify: material specification; manual or machine; position of weld; filler metal specification and classification; shielding gas; single or multiple pass; single or multiple arc; either shielded metal arc, flux cored arc, or gas metal arc welding process; preheat and interpass temperature; welding current; polarity; and root treatment. The welding procedure shall specify welding sequence, pass number, electrode size, welding current amperes and voltage for each joint detail. All the aforementioned information shall be contained on a form that specifies the procedure number, revision number, and the Contractor. The form shall be signed and dated.

Electrodes for manual shielded metal arc welding (SMAW) of Grade 60 steel reinforcing bars shall conform to the requirements of AWS A5.5 of the low hydrogen E90 series.

Solid and composite electrodes for gas metal arc welding (GMAW) and flux-cored arc welding (FCAW) of Grade 60 steel reinforcing bar shall conform to the requirements of AWS A5.28, ER90S and AWS A5.29, E90T respectively. The Contractor shall demonstrate that each combination of electrode and shielding proposed for use will produce the following mechanical properties:

FCAW Grade E90T

Tensile Strength	90,000 psi
Yield Strength	78,000 psi
Elongation in 2-inches	17%

Compliance may be verified from manufacturer's certified test reports, or from actual testing of weld specimens.

All welding shall be protected from air currents, drafts, and precipitation to prevent loss of heat or loss of arc shielding. Short circuiting transfer with gas metal arc welding will not be allowed. Slugging of welds will not be allowed. No field welding of reinforcing bars will be permitted when the ambient temperature is below 32ºF.

The minimum preheat and interpass temperature for welding Grade 60 reinforcing bars shall be in accordance with AWS D1.4 Table 5.2 and mill certification of carbon equivalence, per lot of reinforcing. Preheating shall be applied to the reinforcing bars and other splice members within 6-inches of the weld, unless limited by the available lengths of the bars or splice member.

Generally, post heating of welded splices is only required for direct butt welded splices of Grade 60 bars size No. 9 or larger and shall be done immediately after welding before the splice has cooled to 700ºF. Post heating shall not be less than 800ºF nor more than 1,000ºF and held at this temperature for not less than 10 minutes before allowing the splice to cool naturally to ambient temperature.

Weld joint and welder qualifications shall be made by the following procedures. The joint qualification and welder qualification shall be according to the following tests.

Under supervision of the State Materials and Fabrication Inspector, the welder shall weld three test joints of the largest size reinforcing bar to be weld spliced, per type of joint shown in the Plans. Two of the test welds shall be test loaded to no less than 125 percent of the minimum specified yield strength of the bar. The remaining test weld shall be mechanically cut perpendicular to the direction of the welding and macroetched. The macroetch specimen for Flare V groove welds will be inspected for the weld size and effective throat as shown in the Plans. Indirect butt splices shall be cut mechanically at two locations to provide a transverse cross-section of each of the bars spliced in the test assembly. The sections shall show the full cross-section of the weldment, the root of the weld, and any reinforcement. The etched cross-section shall have complete penetration and complete fusion with the base metal and between successive passes in the weld. Groove welds of direct butt splices and flare-groove welds shall not have reinforcement exceeding $^{1}/_{8}$-inch in height measured from the main body of the bar and shall have a gradual transition to the base metal surface. No cracks will be allowed in either the weld metal or heat-affected zone. All craters shall be filled to the full cross-section of the weld. Weld metal shall be free from overlay. Undercutting deeper than $^{1}/_{32}$-inch will not be allowed except at points where welds intersect the raised pattern of deformations where undercutting less than $^{1}/_{16}$-inch deep will be acceptable. The sum of diameters of piping porosity in groove welds shall not exceed $^{1}/_{8}$-inch in any linear inch of weld or exceed $^{9}/_{16}$-inch in any 6-inch length of weld. Corrections to welds with shielded metal arc, gas metal arc, or flux-cored arc welding processes shall be made in accordance with Engineer's approval

A welder qualified in the vertical position shall then be qualified for the horizontal and flat positions. A welder qualified for the horizontal position shall then be qualified for the flat position but not the vertical position. A welder qualified in the flat position shall be qualified for the flat position only.

Welders qualified for direct butt splice groove welds are qualified for indirect butt splice groove welds and fillet welds. A welder qualified for indirect butt splice grooved welds is not qualified for direct butt splice welds. The welder qualifications shall remain in effect indefinitely unless, (1) the welder is not engaged in a given process of welding for which he/she is qualified for a period exceeding six months, or (2) there is some specific reason to question a welder's ability.

Weld joint geometry shall be as shown in the Plans and these Specifications. Welding machines shall be D.C. current, reverse polarity, and in good working condition.

The Contractor is responsible for using a welding sequence that will limit the alignment distortion of the bars due to the effects of welding. The maximum out-of-line permitted will be $^{1}/_{4}$-inch from a 3.5-foot straight-edge centered on the weld and in line with the bar.

The following procedure for welding steel reinforcing bars is recommended:

Sheared bar ends shall be burned or sawed off a minimum of 1/2-inch to completely remove the ruptured portion of the steel shear area prior to welding butt splices. Surfaces to be welded shall be smooth, uniform, and free from fins, tears, cracks, and other defects. Surfaces to be welded and surfaces adjacent to a weld shall also be free from loose or thick scale, slag, rust, moisture, grease, paint, epoxy covering, or other foreign materials. All tack welds shall be within the area of the final weld. No other tack weld will be permitted. Double bevel groove welds require chipping, grinding, or gouging to sound metal at the root of the weld before welding the other side. Progression of vertical welding shall be upward. The ground wire from the welding machine shall be clamped to the bar being welded.

Should the Contractor elect to use a procedure which differs in any way from the procedure recommended, the Contractor shall submit the changes, in writing, to the Engineer for approval. Approved weld procedures shall be strictly followed.

6-02.3(24)F Mechanical Splices

The Contractor shall form mechanical splices with an Engineer-approved system using sleeve filler metal, threaded coupling, or another method that complies with this section.

If necessary to maintain required clearances after the splices are in place, the Contractor shall adjust, relocate, or add stirrups, ties, and bars.

Before splicing, the Contractor shall provide the Engineer with the following information for each shipment of splice material:

1. The type or series identification (and heat treatment lot number for threaded-sleeve splices),
2. The grade and size of bars to be spliced,
3. A manufacturer's catalog with complete data on material and procedures,
4. A written statement from the manufacturer that the material is identical to that used earlier by the Engineer in testing and approving the system design, and
5. A written statement from the Contractor that the system and materials will be used according to the manufacturer's instructions and all requirements of this section.

All splices shall meet these criteria:

1. Tension splices shall develop at least 130 percent of the yield tensile strength specified for the unspliced bar. The ultimate tensile strength of the sleeve shall exceed that of the other parts of the completed splice.
2. AASHTO M 31 bars within a splice sleeve shall not slip more than 0.03-inch for Grade 40 bars, nor more than 0.045-inch for Grade 60 bars. This slippage shall be measured between gage points clear of the splice sleeve. Measurements shall be taken at an initial load of 3,000 psi and again after loading to 90 percent of the minimum specified yield strength for the unspliced bar and then relaxed to 3,000 psi.
3. Maximum allowable bar size:
    a. Mechanical butt splice — No. 14
    b. Mechanical lap splice — No. 6

The Engineer will visually inspect the splices and accept all that appear to conform with the test samples. For sleeve-filler splices, the Engineer will allow voids within the limits on file in the design approval. If the Engineer considers any splice defective, it shall be removed and replaced at the Contractor's expense.

In preparing sleeve-filler metal splices, the Contractor shall:

1. Clean the bar surfaces by: (a) oxyacetylene torch followed by power wire brushing, or (b) abrasive blasting;
2. Remove all slag, mill scale, rust, and other foreign matter from all surfaces within and 2-inches beyond the sleeve;
3. Grind down any projection on the bar that would prevent placing the sleeve;
4. Prepare the ends of the bars as the splice manufacturer recommends and as the approved procedure requires; and
5. Preheat, just before adding the filler, the entire sleeve and bar ends to 300ºF, plus or minus 50ºF. (If a gas torch is used, the flame shall not be directed into the sleeve.)

When a metallic, sleeve-filler splice is used (or any other system requiring special equipment), both the system and the operator shall qualify in the following way under the supervision of the State Materials and Fabrication Inspector. The operator shall prepare six test splices (three vertical, three horizontal) using bars having the same AASHTO Designation and size (maximum) as those to be used in the work. Each test sample shall be 42-inches long, made up of two 21-inch bars joined end-to-end by the splice. The bar alignment shall not deviate more than $^1/_8$-inch from a straight line over the whole length of the sample. All six samples must meet the tensile strength and slip criteria specified in this section.

The Contractor shall provide labor, materials, and equipment for making these test samples at no expense to the Contracting Agency. The Contracting Agency will test the samples at no cost to the Contractor.

6-02.3(24)G Job Control Tests

As the work progresses, the Engineer may require the Contractor to provide a sample splice (thermal or mechanical) to be used in a job control test. The operator shall create this sample on the job site with the Engineer present using bars of the same size as those being spliced in the work. The sample shall comply with all requirements of these Specifications, and is in addition to all other sample splices required for qualification. The Engineer will require no more than two samples on any project with fewer than 200 splices and no more than one sample per 100 splices on any project with more than 200 splices.

6-02.3(24)H Epoxy-Coated Steel Reinforcing Bar

This work is furnishing, fabricating, coating, and placing epoxy-coated steel reinforcing bars as the Plans, these Specifications, and the Special Provisions require. Coating material shall be applied electrostatically, by spraying, or by the fluidized-bed method.

All epoxy-coated bars shall comply with the requirements of Section 9-07. Fabrication may occur before or after coating.

The Contractor shall protect epoxy-coated bars from damage using padded or nonmetallic slings and straps free from dirt or grit. To prevent abrasion from bending or sagging, the Contractor shall lift bundled bars with a strong-back, multiple supports, or a platform bridge. Bundled bars shall not be dropped or dragged. During shop or field storage, bars shall rest on wooden or padded cribbing. The Contractor may substitute other methods for protecting the bars if the Engineer approves. If the Engineer believes the coated bars have been badly damaged, they will be rejected.

Metal chairs and supports shall be coated with epoxy (or another inert coating if the Engineer approves). The Contractor may use other support devices with the Engineer's approval. Plastic coated tie wires (approved by the Engineer) shall be used to protect the coated bars from being damaged during placement.

The bars shall be placed as the Plans require and held firmly in place during placing and setting of the concrete. All bars shall be placed and fastened as specified in Section 6-02.3(24)C.

In the interval between installing coated bars and concreting the deck, the Contractor shall protect the coating from damage that might result from other construction work.

The Engineer will inspect the coated bars after they are placed and before the deck concrete is placed. The Contractor shall patch any areas that show significant damage (as defined below).

Significant damage means any opening in the coating that exposes the steel in an area that exceeds:

1. 0.05 square inch (approximately $^1/_4$-inch square or $^1/_4$-inch in diameter or the equivalent).
2. 0.012 square inches (approximately $^1/_8$-inch square or $^1/_8$-inch in diameter) when the opening is within $^1/_4$-inch of another opening of equal or larger size.
3. 6-inches long, any width.
4. 0.50 square inch aggregate area in any 1-foot length of bar.

The Contractor shall patch significantly damaged areas with Engineer-approved patching material obtained from the epoxy resin manufacturer. This material shall be compatible with the coating and inert in concrete. Areas to be patched shall be clean and free of surface contaminants. Patching shall be done before oxidation occurs and according to the resin manufacturer's instructions.

6-02.3(25) Prestressed Concrete Girders

The Contractor shall perform quality control inspection. The manufacturing plant of prestressed concrete girders shall be certified by the Precast/Prestressed Concrete Institute's Plant Certification Program for the type of prestress member to be produced and shall be approved by WSDOT as a Certified Prestress Concrete Fabricator prior to the start of production. WSDOT certification will be granted at, and renewed during, the annual prestressed plant review and approval process.

Prior to the start of production of girders, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the work. If the Inspector observes any nonspecification work or unacceptable quality control practices, the Inspector will advise the plant manager. If the corrective action is not acceptable to the Engineer, the girder(s) will be subject to rejection by the Engineer.

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the work and verify the quality of that work. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and workmanship.

The various types of girders are:

Prestressed Concrete Girder – Refers to prestressed concrete girders of all types, including prestressed concrete I girders, prestressed concrete wide flange I girders, bulb tee girders, deck bulb tee girders, thin flange deck bulb tee girders, precast prestressed concrete members, spliced prestressed concrete girders, and prestressed concrete tub girders.

Prestressed Concrete I Girder – Refers to a prestressed concrete girder with a flanged I shaped cross section, requiring a cast-in-place concrete deck to support traffic loads. WSDOT standard girders in this category include Series W42G, W50G, W58G, and W74G.

Prestressed Concrete Wide Flange I Girder – Refers to a prestressed concrete girder with an I shaped cross section with wide top and bottom flanges, requiring a cast-in-place concrete deck to support traffic loads. WSDOT standard girders in this category include Series WF42G, WF50G, WF58G, WF74G, W83G, and W95G.

Bulb Tee Girder – Refers to a prestressed concrete girder, with a wide top flange requiring a cast-in-place concrete deck to support traffic loads. WSDOT standard girders in this category include Series W32BTG, W38BTG, and W62BTG.

Deck Bulb Tee Girder – Refers to a bulb tee girder with a top flange designed to support traffic loads, and designed to be mechanically connected at the flange edges to adjacent girders at the job site. Except where specific requirements are otherwise specified for these girders, deck bulb tee girders shall conform to all requirements specified for bulb tee girders. WSDOT standard girders in this category include Series W35DG, W41DG, W53DG, and W65DG.

Thin Flange Deck Bulb Tee Girder – Refers to a bulb tee girder with a top flange width equal to the girder spacing and requiring a cast-in-place concrete deck to support traffic loads. Except where specific requirements are otherwise specified for these girders, thin flange deck bulb tee girders shall conform to all requirements specified for bulb tee girders. WSDOT standard girders in this category include Series W32TFG, W38TFG, W50TFG, and W62TFG.

Precast Prestressed Member (PCPS Member) – Refers to a precast prestressed slab, precast prestressed ribbed section, or a deck double tee girder. PCPS members are designed to be mechanically connected at the flange or member edges to adjacent PCPS members at the job site. Except where specific requirements are otherwise specified for these girders, PCPS members shall conform to all requirements specified for deck bulb tee girders.

Spliced Prestressed Concrete Girder – Refers to prestressed concrete girders initially fabricated in segments to be longitudinally spliced together with cast-in-place concrete closures at the job site. Except where specific requirements are otherwise specified for these girders, spliced prestressed concrete girders shall conform to all requirements specified for prestressed concrete girders. Anchorages shall conform to Sections 6-02.3(26)B, 6-02.3(26)C, and 6-02.3(26)D. Ducts shall conform to the Section 6-02.3(26)E requirements for internal embedded installation, and shall be round, unless the Engineer approves use of elliptical shaped ducts. Duct-wedge plate transitions shall conform to Section 6-02.3(26)E. Prestressing reinforcement shall conform to Section 6-02.3(26)F. WSDOT standard girders in this category include Series WF74PTG, W83PTG, and W95PTG.

Prestressed Concrete Tub Girder – Refers to prestressed concrete trapezoidal box or bathtub girders including those fabricated in segments to be spliced together with cast-in-place concrete closures at the job site. Except where specific requirements are otherwise specified for these girders, prestressed concrete tub girders shall conform to all requirements specified for prestressed concrete girders and spliced prestressed concrete girders. WSDOT standard girders in this category include Series U**G* or Series UF**G*, where U specifies webs without flanges, UF specifies webs with flanges, ** specifies the girder height in inches, and * specifies the bottom flange width in feet.

6-02.3(25)A Shop Plans

The Plans show design conditions and details for prestressed girders. Deviations will not be permitted, except as specifically allowed by these Specifications and by manufacturing processes approved by the annual plant approval process.

Shop plans shall show the size and location of all cast-in holes for installation of deck formwork hangers and/or temporary bracing. Holes for formwork hangers shall match approved deck formwork plans designed in accordance with Section 6-02.3(16). There shall be no field-drilled holes in prestressed concrete girders. Post-tensioning ducts in spliced prestressed concrete girders shall be located so their center of gravity is in accordance with the Plans.

The Contractor shall have the option to furnish Series W74G prestressed concrete girders with minor dimensional differences from those shown in the Plans. The $2^5/_8$-inch top flange taper may be reduced to $1^5/_8$-inches and the bottom flange width may be increased to 2-feet 2-inches. Other dimensions of the girder shall be adjusted as necessary to accommodate the above mentioned changes. Reinforcing steel shall be adjusted as necessary. The overall height and top flange width shall remain unchanged.

If the Contractor elects to provide a prestressed concrete girder with an increased web thickness, shop plans along with supporting design calculations shall be submitted to the Engineer for approval prior to girder fabrication. The girder shall be designed for at least the same load carrying capacity as the girder shown in the Plans. The load carrying capacity of the mild steel reinforcement shall be the same as that shown in the Plans.

The Contractor may alter bulb tee girder dimensions as specified from that shown in the Plans if:

1. The girder has the same or higher load carrying capacity (using current AASHTO Design Specification);
2. The Engineer approves, before the girder is made, complete design calculations for the girder;
3. The Contractor adjusts substructures to yield the same top of roadway elevation shown in the Plans;
4. The depth of the girder is not increased by more than 2-inches and is not decreased;
5. The web thickness is not increased by more than 1-inch and is not decreased;

6. The top flange minimum thickness of the girder is not increased by more than 2-inches, providing the top flange taper section is decreased a corresponding amount;
7. The top flange taper depth is not increased by more than 1-inch; and
8. The bottom flange width is not increased by more than 2-inches.

The Contractor shall provide five copies of the shop plans to the Engineer for approval, except as otherwise noted. Shop drawings for spliced prestressed concrete girders shall conform to Section 6-02.3(26)A, and seven copies of the shop drawings shall be submitted to the Engineer for approval. The shop drawings for spliced prestressed concrete girders shall include all details related to the post-tensioning operations in the field, including details of hardware required, tendon geometry, blockout details, and details of additional or modified steel reinforcing bars required in cast-in-place closures. Approval of shop plans means only that the Engineer accepts the methods and materials. Approval does not imply correct dimensions.

6-02.3(25)B Casting

Before casting girders, the Contractor shall have possession of an approved set of shop drawings. Side forms shall be steel except that cast-in-place concrete closure forms for spliced prestressed concrete girders, interior forms of prestressed concrete tub girders, and end bulkhead forms of prestressed concrete girders may be plywood. Interior voids for precast prestressed slabs with voids shall be formed by either wax soaked cardboard or expanded polystyrene forms. The interior void forms shall be secured in the position as shown in the Plans and shall remain in place.

All concrete mixes to be used shall be pre-approved in the WSDOT plant certification process and must meet the requirements of Section 9-19.1. The temperature of the concrete when placed shall be between 50ºF and 90ºF.

Slump shall not exceed 4-inches for normal concrete nor 7-inches with the use of a high range water reducing admixture, nor 9-inches when both a high range water reducing admixture is used and the water/cement ratio is less than or equal to 0.35. The high range water reducer shall meet the requirements of Sections 9-23.6 and 9-23.7.

Air-entrainment is not required in the concrete placed into prestressed precast concrete girders, including cast-in-place concrete closures for spliced prestressed concrete girders.

No welds will be permitted on steel within prestressed girders. Once the prestressing steel has been installed, no welds or grounds for welders shall be made on the forms or the steel in the girder, except as specified.

The Contractor may form circular block-outs in the girder top flanges to receive falsework hanger rods. These block-outs shall:

1. Not exceed 1-inch in diameter;
2. Be spaced no more than 72-inches apart longitudinally on the girder;
3. Be located 3-inches or more from the outside edge of the top flange on Series W42G, W50G, W58G, girders, and all prestressed concrete tub girders with webs with flanges, 6-inches or more for Series W74G girders, and 7-inches or more for Series WF42G, WF50G, WF58G, WF74G, WF74PTG, W83G, W83PTG, W95G, W95PTG, W32BTG, W38BTG, W62BTG girders and other bulb tee girders.

The Contractor may form circular block-outs in the girder webs to support brackets for roadway slab falsework. These block-outs shall:

1. Not exceed 1-inch in diameter,
2. Be spaced no more than 72-inches apart longitudinally on the girder, and
3. Be positioned so as to clear the girder reinforcing and prestressing steel.

6-02.3(25)C Prestressing

Each stressing system shall have a pressure gauge or load cell that will measure jacking force. Any gauge shall display pressure accurately and readably with a dial at least 6-inches in diameter or with a digital display. Each jack and its gauge shall be calibrated as a unit and shall be accompanied by a certified calibration chart. The Contractor shall provide one copy of this chart to the Engineer. The cylinder extension during calibration shall be in approximately the position it will occupy at final jacking force.

Jacks and gauges shall be recalibrated and recertified:

1. Annually,
2. After any repair or adjustment, and
3. Anytime there are indications that the jack calibration is in error.

The Engineer may use pressure cells to check jacks, gauges, and calibration charts before and during tensioning.

All load cells shall be calibrated and shall have an indicator that shows prestressing force in the strand. The range of this cell shall be broad enough that the lowest 10 percent of the manufacturer's rated capacity will not be used to measure jacking force.

From manufacture to encasement in concrete, all reinforcement used in girders shall be protected against dirt, oil, grease, damage, rust, and all corrosives. If strands in the stressing bed are exposed before they are encased in concrete, the Contractor shall protect them from contamination or corrosion. The protection method requires the Engineer's approval. If steel has been damaged or if it shows rust or corrosion that cannot be fully removed with a soft cloth, it will be rejected.

Post-tensioning of spliced prestressed concrete girders shall conform to Section 6-02.3(26)G, and the following requirements:

1. Before tensioning, the Contractor shall remove all side forms from the cast-in-place concrete closures. From this point until 48 hours after grouting the tendons, the Contractor shall keep all construction and other live loads off the superstructure and shall keep the falsework supporting the superstructure in place.
2. Once the post-tensioning steel is installed, no welds or welding grounds shall be attached to metal forms, structural steel, or steel reinforcing bars of the structural member.
3. The Contractor shall not tension the post-tensioning reinforcement until the concrete in the cast-in-place closures reaches the minimum compressive strength specified in the Plans (or 5,000 psi if the concrete strength is not specified in the Plans). This strength shall be measured with concrete cylinders made of the same concrete and cured under the same conditions as the cast-in-place closures.
4. All post-tensioning shall be completed before placing the sidewalks and barriers on the superstructure.

6-02.3(25)D Curing

During curing, the Contractor shall keep the girder in a saturated curing atmosphere until the girder concrete has reached the required release strength. If the Engineer approves, the Contractor may shorten curing time by heating the outside of impervious forms. Heat may be radiant, convection, conducted steam, or hot air. With steam, the arrangement shall envelop the entire surface with saturated steam. The Engineer will not permit hot air curing until after approving the Contractor's proposed method to envelop and maintain the girder in a saturated atmosphere. Saturated atmosphere means a relative humidity of at least 90 percent. The Contractor shall never allow dry heat to touch the girder surface at any point.

Under heat curing methods, the Contractor shall:

1. Keep all unformed girder surfaces in a saturated atmosphere throughout the curing time;
2. Embed a thermocouple (linked with a thermometer accurate to plus or minus 5ºF) 6 to 8-inches from the top or bottom of the girder on its centerline and near its midpoint;
3. Monitor with a recording sensor (accurate to plus or minus 5ºF) arranged and calibrated to continuously record, date, and identify concrete temperature throughout the heating cycle;
4. Make this temperature record available for the Engineer to inspect;
5. Heat concrete to no more than 100ºF during the first two hours after placing the concrete, and then increase no more than 25ºF per hour to a maximum of 175ºF;
6. Cool concrete, after curing is complete, no more than 25ºF per hour, to 100ºF; and
7. Keep the temperature of the concrete above 60ºF until the girder reaches release strength.

The Contractor may strip side forms from prestressed concrete girders once the concrete has reached a minimum compressive strength of 3,000 psi. All damage from stripping is the Contractor's responsibility.

Curing of cast-in-place concrete closures for spliced prestressed concrete girders shall conform to Section 6-02.3(11).

6-02.3(25)E Contractors Control Strength

Concrete strength shall be measured on test cylinders cast from the same concrete as that in the girder. These cylinders shall be cured under time-temperature relationships and conditions that simulate those of the girder. If the forms are heated by steam or hot air, test cylinders will remain in the coolest zone throughout curing. If forms are heated another way, the Contractor shall provide a record of the curing time-temperature relationship for the cylinders for each girder to the Engineer. When two or more girders are cast in a continuous line and in a continuous pour, a single set of test cylinders may represent all girders provided the Contractor demonstrates uniformity of casting and curing to the satisfaction of the Engineer.

The Contractor shall mold, cure, and test enough of these cylinders to satisfy specification requirements for measuring concrete strength. The Contractor may use 4-inch by 8-inch or 6-inch by 12-inch cylinders. If heat is used to shorten curing time, the Contractor shall let cylinders cool for at least $^1/_2$-hour before testing.

Test cylinders may be cured in a moist room or water tank in accordance with WSDOT FOP for AASHTO T-23 after the girder concrete has obtained the required release strength. If, however, the Contractor intends to ship the girder prior to the standard 28-day strength test, the design strength for shipping shall be determined from cylinders placed with the girder and cured under the same conditions as the girder. These cylinders may be placed in a noninsulated, moisture-proof envelope.

To measure concrete strength in the girder, the Contractor shall randomly select two test cylinders and average their compressive strengths. The compressive strength in either cylinder shall not fall more than 5 percent below the specified strength. If these two cylinders do not pass the test, two other cylinders shall be selected and tested.

If too few cylinders were molded to carry out all required tests on the girder, the Contractor shall remove and test cores from the girder under the surveillance of the Engineer. If the Contractor casts cylinders to represent more than one girder, all girders in that line shall be cored and tested.

For precast prestressed members, a test shall consist of four cores measuring 3-inches in diameter by 6-inches in height (for slabs) and by the thickness of the web (for ribbed sections). Two cores shall be taken from each side of the member and on each side of the member's span midpoint, at locations approved by the Engineer. The core locations for precast prestressed slabs shall be near mid-depth of the slab, within the middle third of the span length, and shall avoid all prestressing strands and steel reinforcing bars. The core locations for precast prestressed ribbed sections shall be immediately beneath the top flange, within the middle third of the span length, and shall avoid all prestressing strands and steel reinforcing bars.

For prestressed concrete tub girders, a test shall consist of four cores measuring 3-inches in diameter by the thickness of the web, taken from each web approximately three feet to the left and to the right of the center of the girder span. The cores shall avoid all prestressing strands and steel reinforcing bars.

For all other prestressed concrete girders, a test shall consist of three cores measuring 3-inches in diameter by the thickness of the web and shall be removed from just below the top flange; one at the midpoint of the girder's length and the other two approximately 3-feet to the left and approximately 3-feet to the right.

The cores shall be taken in accordance with AASHTO T 24 and shall be tested in accordance with WSDOT FOP for AASHTO T 22. The Engineer may accept the girder if the average compressive strength of the four cores from the precast prestressed member, or prestressed concrete tub girder, or of the three cores from any other prestressed concrete girder, is at least 85 percent of the specified compressive strength with no one core less than 75 percent of specified compressive strength.

If the girder is cored to determine the release strength, the required patching and curing of the patch shall be done prior to shipment. If there are more than three holes or if they are not in a neutral location, the prestress steel shall not be released until the holes are patched and the patch material has attained a minimum compressive strength equal to the required release compressive strength or 4,000 psi, whichever is larger.

The Contractor shall coat cored holes with an epoxy bonding agent and patch the holes using the same type concrete as that in the girder, or a mix approved during the annual plant review and approval. The epoxy bonding agent shall meet the requirements of Section 9-26.1 for Type II, Grade 2 epoxy. The girder shall not be shipped until tests show the patch material has attained a minimum compressive strength of 4,000 psi

6-02.3(25)F Prestress Release

Side and flange forms that restrain deflection shall be removed before release of the prestressing reinforcement.

All harped and straight strands shall be released in a way that will produce the least possible tension in the concrete. This release shall not occur until tests show each girder has reached the minimum compressive strength required by the Plans.

The Contractor may request permission to release the prestressing reinforcement at a minimum concrete compressive strength less than specified in the Plans. This request shall be submitted to the Engineer for approval in accordance with Section 6-01.9 and shall be accompanied with calculations showing the adequacy of the proposed release concrete compressive strength. The release strength shall not be less than 3,500 psi, except that the release strength for spliced prestressed concrete girders shall not be less than 4,000 psi. The calculated release strength shall meet the requirements outlined in the Washington State Department of Transportation Bridge Design Manual for tension and compression at release. The proposed minimum concrete compressive strength at release will be evaluated by the Contracting Agency. Fabrication of girders using the revised release strength shall not begin until the Contracting Agency has provided written approval of the revised release compressive strength. If a reduction of the minimum concrete compressive strength at release is allowed, the Contractor shall bear any added cost that results from the change.

6-02.3(25)G Protection of Exposed Reinforcement

When a girder is removed from its casting bed, all bars and strands projecting from the girder shall be cleaned and painted with a minimum dry film thickness of 1 mil of paint Formula No. A-9-73. During handling and shipping, projecting reinforcement shall be protected from bending or breaking. Just before placing concrete around the painted projecting bars or strands, the Contractor shall remove from them all dirt, oil, and other foreign matter.

Grouting of post-tensioning ducts for spliced prestressed concrete girders shall conform to Section 6-02.3(26)H.

6-02.3(25)H Finishing

The Contractor shall apply a Class 1 finish, as defined in Section 6-02.3(14), to:

1. The exterior surfaces of the outside girders;
2. The bottoms, sides, and tops of the lower flanges on all girders; and

All other girder surfaces shall receive a Class 2 finish.

The interface on I-girders and other girders that contact the cast-in-place deck shall have a finish of dense, screeded concrete without a smooth sheen or laitance on the surface. After vibrating and screeding, and just before the concrete reaches initial set, the Contractor shall texture the interface. This texture shall be applied with a steel brooming tool that etches the surface transversely leaving grooves $^1/_8$-inch to $^1/_4$-inch wide, between $^1/_8$-inch and $^1/_4$-inch deep, and spaced $^1/_4$-inch to $^1/_2$-inch apart.

On the deck bulb tee girder section and all precast prestressed members, the Contractor shall test the roadway deck surface portion for flatness. This test shall occur after floating but while the concrete remains plastic. Testing shall be done with a 10-foot straightedge parallel to the girder centerline and with a flange width straightedge at right angles to the girder centerline. The Contractor shall fill depressions, cut down high spots, and refinish to correct any deviation of more than $^1/_4$-inch within the straightedge length.

This section of the roadway surface shall be finished to meet the requirements for finishing roadway slabs, as defined in Section 6-02.3(10) except that, if approved by the Engineer, a coarse stiff broom may be used to provide the finish in lieu of a metal tined comb.

The Contractor may repair rock pockets and other defects in the girder provided the repair is covered in the annual plant approval package. All other repairs and repair procedures shall be documented and approved by the Engineer prior to acceptance of the girder.

6-02.3(25)I Fabrication Tolerances

The girders shall be fabricated as shown in the Plans and shall meet the dimensional tolerances listed below. Construction tolerances of cast-in-place closures for spliced prestressed concrete girders shall conform to the tolerances specified for spliced prestressed concrete girders. Actual acceptance or rejection will depend on how the Engineer believes a defect outside these tolerances will affect the structure's strength or appearance:

1. Prestressed Concrete Girder Length (overall): ± 1/4-inch per 25-feet of beam length, up to a maximum of ± 1-inch.
2. Precast Prestressed Member Length (overall): ± 1-inch.
3. Width (flanges): + 3/8-inch, - 1/4-inch.
4. Width (narrow web section): + 3/8-inch, - 1/4-inch.
5. Width (Precast Prestressed Member): ± 1/4-inch.
6. Girder Depth (overall): ± 1/4-inch.
7. Flange Depth:

For I and Wide Flange I girders:	± 1/4-inch
For bulb tee and deck bulb tee girders:	+ 1/4-inch, - 1/8-inch
For PCPS members:	+ 1/4-inch, - 1/8-inch

8. Strand Position in Prestressed Concrete Girder: ± 1/4-inch from the center of gravity of an individual strand; ± 1/2-inch from the center of gravity of a bundled strand group.
9. Strand Position in Precast Prestressed Member: ±1/4-inch from the center of gravity of a bundled strand group and of an individual strand.
10. Longitudinal Position of the Harping Point:

Single harping point	± 18-inches
Multiple bundled strand groups	
First bundled strand group	± 6-inches
Second bundled strand group	± 18-inches
Third bundled strand group	± 30-inches

11. Position of an interior void, vertically and horizontally (Precast Prestressed Slab with voids): ± 1/2-inch.
12. Bearing Recess (center recess to beam end): ± 1/4-inch.
13. Beam Ends (deviation from square or designated skew):
    Horizontal: ± 1/2-inch from web centerline to girder flange
    Vertical: ± 1/8-inch per foot of beam depth

14. Precast Prestressed Member Ends (deviation from square or designated skew): ± $^{1}/_{2}$-inch.
15. Bearing Area Deviation from Plane (in length or width of bearing): $^{1}/_{16}$-inch.
16. Stirrup Reinforcing Spacing: ± 1-inch.
17. Stirrup Projection from Top of Beam: ± $^{3}/_{4}$-inch.
18. Mild Steel Concrete Cover: - $^{1}/_{8}$-inch, + $^{3}/_{8}$-inch.
19. Offset at Form Joints (deviation from a straight line extending 5-feet on each side of joint): ± $^{1}/_{4}$-inch.
20. Deviation from Design Camber (Precast Prestressed Member): ± $^{1}/_{4}$-inch per ten feet of member length measured at midspan, but not greater than ± $^{3}/_{4}$-inch total.
21. Differential Camber Between Girders in a Span (measured in place at the job site):

For I, Wide Flange I, bulb tee, and spliced prestressed concrete girders:	$^{1}/_{8}$-inch per 10-feet of beam length.
For deck bulb tee girders:	Cambers shall be equalized by an approved method when the differences in cambers between adjacent girders or stages measured at mid-span exceeds $^{1}/_{4}$-inch.
For PCPS members:	± $^{1}/_{4}$-inch per ten feet of member length measured at midspan, but not greater than ± $^{1}/_{2}$-inch total.
For prestressed concrete tub girders:	± $^{1}/_{4}$-inch per ten feet of member length measured at midspan, but not greater than ± $^{1}/_{3}$-inch total.

22. Position of Inserts for Structural Connections: ± 1-inch.
23. Position of Lifting Loops: ± 3-inches longitudinal, ± 1-inch transverse.
24. Weld plates for bulb tee girders shall be placed ± $^{1}/_{2}$-inch longitudinal and ± $^{1}/_{8}$-inch vertical.
25. Position of post-tensioning ducts at girder and CIP closure ends: ± $^{1}/_{4}$-inch.
26. Position of post tensioning ducts along segments of segmental prestressed concrete girders: ± $^{1}/_{4}$-inch.
27. Deviation from a smooth curve for post-tensioning ducts at closures based on the sum total of duct placement and alignment tolerances: ± $^{3}/_{8}$-inch.

6-02.3(25)J Horizontal Alignment

The Contractor shall check and record the horizontal alignment of both top and bottom flanges of each girder upon removal of the girder from the casting bed. The Contractor shall also check and record the horizontal alignment within a two-week period prior to shipment, but no less than three days prior to shipment. If the girder remains in storage for a period exceeding 120 days, the Contractor shall check and record the horizontal alignment at approximately 120 days. Each check shall be made by measuring the distance between each flange and a chord that extends the full length of the girder. The Contractor shall perform and record each check at a time when the alignment of the girder is not influenced by temporary differences in surface temperature. These records shall be available for the Engineer's inspection and included in the Contractor's Prestressed Concrete Certificate of Compliance.

Immediately after the girder is removed from the casting bed, neither flange shall be offset more than 1/8-inch for each 10-feet of girder length. During storage and prior to shipping, the offset (with girder ends plumb and upright and with no external force) shall not exceed 1/4-inch per 10-feet of girder length. Any girder within this tolerance may be shipped, but must be corrected at the job site to the 1/8-inch maximum offset per 10-feet of girder length before concrete is placed into the diaphragms.

The Engineer may permit the use of external force to correct girder alignment at the plant or job site if the Contractor provides stress calculations and a proposed procedure. If external force is permitted, it shall not be released until after the roadway slab has been placed and cured ten days.

The maximum deviation of the side of the precast prestressed slab, or the edge of the roadway deck slab of the deck double tee girder or the precast prestressed ribbed section, measured from a chord that extends end to end of the member, shall be ± 1/8-inch per 10-feet of member length, but not greater than 1/2-inch total.

A final alignment check shall be performed within three days prior to shipment to the jobsite. All precast prestressed members which exceed the specified horizontal alignment tolerance may be subject to rejection.

6-02.3(25)K Girder Deflection

The Contractor shall check and record the vertical deflection (camber) of each girder upon removal of the girder from the casting bed. If the girder remains in storage for a period exceeding 120 days, the Contractor shall check and record the vertical deflection (camber) within a two-week period prior to shipment, but no less than three days prior to shipment. The Contractor shall perform and record each check at a time when the alignment of the girder is not influenced by temporary differences in surface temperature. These records shall be available for the Engineer's inspection, and in the case of girders older than 120 days, shall be transmitted to the Engineer as soon as practical for evaluation of the effect of long-term storage on the "D" dimension. These records shall also be included in the Contractor's Prestressed Concrete Certificate of Compliance.

The "D" dimensions shown in the Plans are computed girder deflections at midspan based on a time lapse of 40 and 120 days after release of the prestressing strands. A positive (+) "D" dimension indicates upward deflection.

The Contractor shall control the deflection of prestressed concrete girders that are to receive a cast-in-place slab by scheduling fabrication between 40 and 120 days of girder erection.

If it is anticipated that the girders will be older than 120 days at the time of erection, the Contractor shall submit calculations to the Engineer showing the estimated girder deflection at midspan at the age anticipated for erection. This submittal shall also include the Contractor's proposal for accommodating any excess camber in the construction. The Contractor shall not proceed with girder fabrication until this submittal is approved by the Engineer. The actual girder deflection at the midspan may vary from the "D" dimension at the time of slab forming by a maximum of plus 1/2-inch for girder lengths up to 80-feet, and plus 1-inch for girder lengths over 80-feet, but less than or equal to 140-feet, and plus 1 1/2-inches for girder lengths over 140-feet.

All costs, including roadway slab form adjustments required to maintain specified steel reinforcing bar clearances and deck profiles, and any additional Contracting Agency engineering expenses, in connection with accommodating excess girder deflection shall be at the Contractor's expense.

6-02.3(25)L Handling and Storage

During handling and storage, each girder shall always be kept plumb and upright, and each precast prestressed member and prestressed concrete tub girder shall always be kept in the horizontal position as shown in the Plans. It shall be lifted only by the lifting devices (strand lift loops or high-strength threaded steel bars) at either end. For strand lift loops, a minimum 2-inch diameter straight pin of a shackle shall be used through the loops. For high-strength threaded steel bars, the lifting hardware that connects to the bars shall be designed, detailed, and furnished by the Contractor. Series W42G, WF42G, W50G, WF50G, W58G, and WF58G girders, and Series W32BTG, W38BTG, W62BTG, and W74G girders up to 145-feet in length, can be picked up at a minimum angle of 60 degrees from the top of the girder. All other prestressed girders shall be picked up within 10 degrees of perpendicular to the top of the girder.

For some girders, straight temporary top flange strands may be specified in the Plans. Pretensioned top temporary strands for full length prestressed concrete girders shall be unbonded over all but the end 10-feet of the girder length. As an alternative for full length prestressed concrete girders, temporary top strands may be post-tensioned prior to shipment. When temporary top strands are specified for spliced prestressed concrete girders, the temporary top strands shall be post-tensioned prior to lifting the assembled girder. When the post-tensioned alternative is used, the Contractor shall be responsible for properly sizing the anchorage plates, and the reinforcement adjacent to the anchorage plates, to prevent bursting or splitting of the concrete in the top flange. Temporary strands shall be cut or released in accordance with Section 6-02.3(25)N.

The Contractor may request permission to use lifting devices, lifting device locations, lifting angles, concrete release strengths, or temporary top strand configurations other than specified in the Plans. The number of temporary top strands may be increased from the number shown in the Plans but shall not be decreased. The request, including calculations showing the adequacy of the proposed lifting method, shall be submitted to the Engineer for approval in accordance with Section 6-01.9. The Contractor's analysis shall conform to Article 5.2.9 of the *PCI Design Handbook, Precast and Prestressed Concrete*, Fifth Edition, or other approved methods. The Contractor's calculations shall verify that the concrete stresses in the prestressed girder do not exceed those listed in Section 6-02.3(25)M. The Contractor shall not begin girder lifting operations under the provisions of the lifting method submittal until receiving the Engineer's written approval of the submittal, and shall perform the girder lifting operations at no additional expense to the Contracting Agency.

If girders are to be stored, the Contractor shall place them on a stable foundation that will keep them in a vertical position. Stored girders shall be supported at the bearing recesses or, if there are no recesses, approximately 18-inches from the girder ends. After prestressing, precast prestressed members shall be supported at points between 1′-0" and 2′-0" from the member ends. After post-tensioning, segmental prestressed concrete girders shall be supported at points between 2′-0" and 5′-0" from the girder ends, unless otherwise shown in the Plans. For long-term storage of girders with initial horizontal curvature, the Contractor may wedge one side of the bottom flange, tilting the girders to control curvature. If the Contractor elects to set girders out of plumb during storage, the Contractor shall have the proposed method analyzed by the Contractor's engineer to ensure against damaging the girder.

6-02.3(25)M Shipping

After the girder has reached its 28-day design strength, and the fabricator believes it to comply with the specification, the girder and a completed Certification of Compliance, signed by a Precast/Prestressed Concrete Institute Certified Technician or a professional engineer, acceptable to the Contracting Agency, shall be submitted to the Engineer for inspection. If the Engineer finds the certification and the girder to be acceptable, the Engineer will stamp the girder "Approved for Shipment."

No double tee girder, deck double tee girder, precast prestressed slab or precast prestressed ribbed section shall be shipped for at least three days after concrete placement. No deck bulb tee girder or prestressed concrete tub girder shall be shipped for at least seven days after concrete placement, except that deck bulb tee girders or prestressed concrete tub girders may be shipped three days after concrete placement when L/(bd) is less than or equal to 5.0, where L equals the shipping length of the girder, b equals the girder top flange width (for deck bulb tee girders) or the bottom flange width (for prestressed concrete tub girders), and d equals the girder depth, all in feet. No other girder shall be shipped for at least ten days after concrete placement.

Girder support during shipping shall be located as follows unless otherwise shown in the Plans:

Type of Girder	Centerline Support Within This Distance From Either End
Precast Prestressed Members	2-feet
Series W42G, WF42G, W50G and WF50G	3-feet
All bulb tee and deck bulb tee girders, except as noted	3-feet
Series W58G, WF58G, and W62BTG	4-feet
Series W74G and WF74G	5-feet
Series W83G and W95G	8-feet
Series WF74PTG, W83PTG, and W95PTG segments	8-feet
Prestressed concrete tub girder segment	4-feet

The Contractor may request permission to use support locations other than those specified. The Contractor shall submit the support location modification proposal, with supporting calculations, to the Engineer for approval in accordance with Section 6-01.9. If the support locations are moved closer to the lateral ends of the girders, the calculations shall demonstrate adequate control of lateral bending during shipping. The calculations shall also show that concrete stresses in the girders will not exceed those listed below.

If the Contractor elects to assemble spliced prestressed concrete girders into components of two or more segments prior to shipment, the Contractor shall submit shipment support location working drawings with supporting calculations to the Engineer in accordance with Section 6-01.9. The calculations shall show that concrete stresses in the assembled girders will not exceed those listed below.

Lateral bracing for shipping is not required for prestressed concrete tub girders and precast prestressed members. Other prestressed concrete girders of lengths equal or shorter than the following will not require lateral bracing for shipping:

Type of Girder	Maximum Length Not Requiring Bracing for Shipping
Series W42G, WF42G, W32BTG, and W38BTG	80-feet
Series W50G and WF50G	100-feet
Series W58G, WF58G, and W62BTG	105-feet
All deck bulb tee girders	120-feet
Series W74G and WF74G	130-feet

For all girders exceeding these lengths, and all Series WF74PTG, W83G, W83PTG, W95G, and W95PTG girders, the Contractor shall provide bracing to control lateral bending during shipping, unless the Contractor furnishes calculations in accordance with Section 6-01.9 demonstrating that bracing is not necessary. External bracing shall be attached securely to the top flange of the girder. The Contractor is cautioned that more conservation guidelines for lateral bracing may be required for some delivery routes. The Contractor shall submit a bracing plan, with supporting calculations, to the Engineer for approval in accordance with Section 6-01.9. The Contractor shall not begin shipping the girders until receiving the Engineer's approval of the bracing plan, and shall perform all bracing operations at no additional cost to the Contracting Agency.

Criteria for Checking Girder Stresses

At the Time of Lifting or Transporting and Erecting

Stresses at both support and harping points shall be satisfied based on these criteria:

1. Allowable compression stress, fc = 0.60f'cm
   a. f'cm = compressive strength at time of lifting or transporting verified by test but shall not exceed design compressive strength (f'c) at 28 days in psi + 1,000 psi
2. Allowable tension stress, ksi
   a. With no bonded reinforcement = 3 times square root (f'cm) ≤ 0.20 ksi
   b. With bonded reinforcement to resist total tension force in the concrete computed on the basis of an uncracked section 6.0 times square root (f'cm). The allowable tensile stress in the reinforcement is 30 ksi (AASHTO M-31, Gr. 60)
3. Prestress losses
   a. 1 day to 1 month = computed losses
   b. 1 month to 1 year = 75 percent of computed final losses
   c. 1 year or more = computed final losses
4. Impact on dead load
   a. Lifting from casting beds = 0 percent
   b. Transporting and erecting = 20 percent

6-02.3(25)N Prestressed Concrete Girder Erection

Before beginning to erect any prestressed concrete girders, the Contractor shall submit to the Engineer for review and shall have received approval for the erection plan and procedure describing the methods the Contractor intends to use. The erection plan and procedure shall provide complete details of the erection process including but not limited to:

1. Temporary falsework support, bracing, guys, deadmen, and attachments to other structure components or objects;
2. Procedure and sequence of operation;
3. Girder stresses during progressive stages of erection;
4. Girder weights, lift points, lifting devices, spreaders, and angle of lifting cables in accordance with Section 6-02.3(25)L, etc.;
5. Crane(s) make and model, mass, geometry, lift capacity, outrigger size and reactions;
6. Girder launcher or trolley details and capacity (if intended for use); and
7. Locations of cranes, barges, trucks delivering girders, and the location of cranes and outriggers relative to other structures, including retaining walls and wing walls.

The erection plan shall include drawings, notes, catalog cuts, and calculations clearly showing the above listed details, assumptions, and dimensions. Material properties and specifications, structural analysis, and any other data used shall also be included. The plan shall be prepared by (or under the direct supervision of) a Professional Engineer, licensed under Title 18 RCW, State of Washington, in the branch of Civil or Structural, and shall carry the engineer's seal and signature, in accordance with Section 6-02.3(16).

The Contractor shall submit the erection plans, calculations, and procedure directly to the Bridge and Structures Office, Construction Support Engineer, in accordance with Section 6-02.3(16). After the plan is approved and returned to the Contractor, all changes that the Contractor proposes shall be submitted to the Engineer for review and approval.

When prestressed girders arrive on the project, the Project Engineer will confirm that they are stamped "Approved for Shipment" and that they have not been damaged in shipment before accepting them.

The concrete in piers and crossbeams shall reach at least 80 percent of design strength before girders are placed on them. The Contractor shall hoist girders only by the lifting devices at the ends, always keeping the girders plumb and upright. Once erected, the girders shall be braced to prevent tipping until the intermediate diaphragms are cast and cured. **When temporary strands in the top flange are used, they shall be cut after the girders are braced and before the intermediate diaphragms are cast**. The Contractor shall place the cast-in-place deck on the girders within 30 calendar days of cutting the temporary strands, except as otherwise approved by the Engineer.

For situations where the Contractor proposes to delay placing the cast-in-place deck on the girders beyond 30 calendar days after cutting the temporary strands, the Contractor shall submit supporting girder camber calculations to the Engineer for approval in accordance with Section 6-01.9. The Contractor shall not cut the temporary strands until receiving the Engineer's approval of the girder camber calculations.

Instead of the oak block wedges shown in the Plans, the Contractor may use Douglas fir blocks if the grain is vertical.

The Contractor shall check the horizontal alignment of both the top and bottom flanges of each girder after girder erection but before placing concrete in the bridge diaphragms as described in Section 6-02.3(25)J.

The Contractor shall fill all block-out holes and patch any damaged area caused by the Contractor's operation, with an approved mix, to the satisfaction of the Engineer.

For precast prestressed concrete slabs, the Contractor shall place the 1¼-inch diameter vertical dowel bars at the top of the pier walls as shown in the Plans. The Contractor shall either form the hole or core drill the hole following the alternatives shown in the Plans. The portion of the dowel bar in the top of the pier walls shall be set with either grout that complies with Section 9-26.3 or type II epoxy bonding agent conforming to Section 9-26.1 following placement of each precast prestressed slab.

6-02.3(25)O Deck Bulb Tee Girder Flange Connection

The Contractor shall submit a method of equalizing deck bulb tee girder (and precast prestressed member) deflections to the Engineer for approval in accordance with Section 6-01.9, except that the submittal shall be made a minimum of 60 days prior to field erection of the deck bulb tee girder. Deflection equalizing methods approved for previous Contracting Agency contracts will be acceptable providing the bridge configuration is similar and the previous method was satisfactory. A listing of the previous Contracting Agency contract numbers for which the method was used shall be included with the submittal. The weld-ties may be used as a component of the equalizing system provided the Contractor's procedure outlines how the weld-ties are to be used, and that the Contractor's submittal includes a list and description of previous bridge projects where the Contractor has successfully used weld-ties as a component of the equalizing system.

The concrete diaphragms for deck bulb tee girders shall attain a minimum compressive strength of 2,500 psi before any camber equalizing equipment is removed.

On deck bulb tee girders, girder deflection shall be equalized utilizing the approved method before girders are weld-tied and before keyways are filled. Keyways between tee girders shall be filled flush with the surrounding surfaces with nonshrink grout, except that keyways for deck bulb tee girders receiving a cast-in-place concrete deck slab need not be filled with grout. This nonshrink grout shall have a compressive strength of 5,000 psi before the equalizing equipment is removed. Compressive strength shall be determined by fabricating and testing cubes in accordance with WSDOT Test Method 813 and testing in accordance with WSDOT FOP for AASHTO T-106.

Welding ground shall be attached directly to the steel plates being welded when welding the weld-ties on bulb tee girders.

No construction equipment shall be placed on the structure, other than equalizing equipment, until the girders have been weld-tied and the keyway grout has attained a compressive strength of 5,000 psi.

6-02.3(26) Cast-in-Place Prestressed Concrete

Unless otherwise shown in the Plans, concrete for cast-in-place prestressed bridge members shall be Class 4000D in the roadway deck, and Class 4000 at all other locations. Air entrainment shall conform to Sections 6-02.3(2)A and 6-02.3(3).

The Contractor shall construct supporting falsework in a way that leaves the superstructure free to contract and lift off the falsework during post-tensioning. Forms that will remain inside box girders to support the roadway slab shall, by design, resist girder contraction as little as possible.

Before tensioning, the Contractor shall remove all side forms from girders. From this point until 48 hours after grouting the tendons, the Contractor shall keep all construction and other live loads off the superstructure and shall keep the falsework supporting the superstructure in place.

Once the prestressing steel is installed, no welds or welding grounds shall be attached to metal forms, structural steel, or reinforcing bars of the structural member.

The Contractor shall not stress the strands until all concrete has reached a compressive strength of at least 4,000 psi (or the strength shown in the Plans). This strength shall be measured on concrete test cylinders made of the same concrete cured under the same conditions as the cast-in-place unit.

All post-tensioning shall be completed before sidewalks and barriers are placed.

6-02.3(26)A Shop Drawings

Before casting the structural elements, the Contractor shall submit:

1. Seven sets of shop drawings for approval by the Bridge and Structures Engineer.

   US Postal Service:

   Washington State Department of Transportation
   Bridge and Structures Engineer
   Construction Support
   PO Box 47340
   Olympia, WA 98504-7340

   FedEx:

   Washington State Department of Transportation
   Bridge and Structures Engineer
   Construction Support
   4500 3rd Avenue SE
   Lacey, WA 98503
2. Two sets of shop drawings to the Project Engineer.

These shop drawings shall show complete details of the methods, materials, and equipment the Contractor proposes to use in prestressing work. The shop drawings shall follow the design conditions shown in the Plans unless the Engineer permits equally effective variations.

In addition, the shop drawings shall show:

1. The method and sequence of stressing.
2. Technical data on tendons and steel reinforcement, anchoring devices, anchoring stresses, types of tendon conduit, and all other data on prestressing operations.
3. Stress and elongation calculations. Separate stress and elongation calculations shall be submitted for each tendon if the difference in tendon elongations exceeds 2 percent.
4. That tendons in the bridge will be arranged to locate their center of gravity as the Plans require.
5. Details of additional or modified reinforcing steel required by the stressing system.
6. Procedures and lift-off forces at both ends of the tendon for performing a force verification lift-off in the event of discrepancies between measured and calculated elongations.

Couplings or splices will not be permitted in prestressing strands. Couplings or splices in bar tendons are subject to the Engineer's approval.

Friction losses used to calculate forces of the post-tensioning steel shall be based on the assumed values used for the design. The assumed anchor set, friction coefficient "μ", and friction wobble coefficient "k" values for design are shown in the Plans. The post-tensioning supplier may revise the assumed anchor set value provided all the stress and force limits listed in Section 6-02.3(26)E are met.

The Contractor shall determine all points of interference between the mild steel reinforcement and the paths of the post-tensioning tendons. Details to resolve interferences shall be submitted with the shop drawings for approval. Where reinforcing bar placement conflicts with post-tensioning tendon placement, the tendon profile shown in the Plans shall be maintained. Mild steel reinforcement for post-tensioning anchorage zones shall not be fabricated until after the post-tensioning shop drawings have been approved by the Engineer.

Approval of these shop drawings will mean only that the Engineer considers them to show a reasonable approach in enough detail. Approval will not indicate a check on dimensions.

The Contractor may deviate from the approved shop drawings only after obtaining the Engineer's approval of a written request that describes the proposed changes. Approval of a change in method, material, or equipment shall not relieve the Contractor of any responsibility for completing the work successfully.

Before physical completion of the project, the Contractor shall provide the Engineer with reproducible originals of the shop drawings (and any approved changes). These shall be clear, suitable for microfilming, and on permanent sheets that measure no smaller than 11 by 17-inches. Alternatively, the shop drawings may be provided in an electronic format with the approval of the Bridge and Structures Engineer.

6-02.3(26)B General Requirements for Anchorages

Post-tensioning reinforcement shall be secured at each end by means of an approved anchorage device, which shall not kink, neck down, or otherwise damage the post-tensioning reinforcement. The anchorage assembly shall be grouted to the Engineer's satisfaction.

The structure shall be reinforced with steel reinforcing bars in the vicinity of the anchorage device. This reinforcement shall be categorized into two zones. The first or local zone shall be the anchorage region that closely surrounds the specific anchorage device. The second or general zone shall be the portion of the anchorage region more remote from the immediate anchorage device.

The steel reinforcing bars required locally for the concrete confinement immediately around the anchorage device (first or local zone) shall be calculated by the post-tensioning system supplier and shall be shown in the shop drawings. The calculations shall be submitted with the shop drawings. The first or local zone steel reinforcing bars shall be furnished and installed by the Contractor, at no additional cost to the Contracting Agency, in addition to the structural reinforcement required by the Plans. The steel reinforcing bars required in the second or general zone shall be as shown in the Plans and are included in the appropriate bid items.

The Contractor shall submit details, certified test reports, and/or supporting calculations, as specified below, which verify the structural adequacy of the anchorage devices for approval by the Engineer. This requirement does not apply where the anchorage devices have been previously approved by the Contracting Agency for the same structure configuration. The Contractor shall also submit any necessary changes to the Contract Plans. The test report shall specify all pertinent test data.

Dead ended anchorages will not be permitted. Dead ended anchorages are defined as anchorages that cannot be accessed during the stressing operations.

Materials and workmanship shall conform to the applicable requirements of Sections 6-03 and 9-06.

Before installing the anchorage device, the Contractor shall submit to the Engineer a Manufacturer's Certificate of Compliance in accordance with Section 1-06.3.

The Contractor's proposed anchorage devices shall meet the requirements listed in either Sections 6-02.3(26)C or 6-02.3(26)D.

6-02.3(26)C Bearing Type Anchorages

Bearing type anchorages shall conform to the following requirements:

1. The allowable bearing stress under P_{jack} prior to seating shall be taken:
   a. If $\rho_s = 0$ percent then $f_{cpi} = 0.5\ f'_{ci}\ (A/A_g)^{1/2} < f'_{ci}$
   b. If $\rho_s \geq 2$ percent then $f_{cpi} = 0.75\ f'_{ci}\ (A/A_g)^{1/2} < 1.5\ f'_{ci}$

   For ρ_s between 0 percent and 2 percent the allowable bearing stress may be linearly interpolated.

   For lightweight concrete the allowable bearing stress shall be reduced by 20 percent.
2. The average concrete bearing stress on the net bearing area at the time of jacking shall not exceed:

   $f_{bi} = P_{jack}/A_{net} < f_{cpi}$
3. The bending stress in bearing plate at P_{jack} shall not exceed:

   $f_s = 3f_{bi}\ (n/t)^2 < 0.8f_{sy}$

   with stiffness $n/t < 0.08\ (E_s/f_{bi})^{1/3}$
4. The allowable bearing stress between bearing plate and wedge plate at P_{jack} shall not exceed:

   $f_{sbi} < 1.5\ f_{sy}$

where:

P_{jack}	=	Jacking force, but not less than 80 percent MUTS
MUTS	=	Acronym for Minimum Ultimate Tensile Strength, MUTS is the force equal to the nominal cross sectional area of strand, or bar, times their nominal tensile stress
AUTS	=	Acronym for Actual Ultimate Tensile Strength, measured as a force
a_x	=	Dimension of distribution area in X direction
a_y	=	Dimension of distribution area in Y direction
$A = a_x\ a_y$	=	Distribution area within concrete support area
b_x	=	Dimension of bearing plate in X direction
b_y	=	Dimension of bearing plate in Y direction

A_b	=	Net bearing area
A_{net}	=	Net bearing plate area after deducting center hole area
$A_g = b_x\ b_y$	=	Gross bearing area
e_{max}, e_{min}	=	Maximum and minimum edge cover of bearing plate in distribution area
f'_{ci}	=	Compressive strength of concrete at time of initial stressing
f'_{cpi}	=	Permissible concrete compressive strength at time of jacking
f_{bi}	=	Average uniform concrete bearing stress under bearing plate prior to seating produced by P_{jack}
f'_c	=	Compressive strength of concrete at 28 days
f'_s	=	Bending stress in steel bearing plate
f_{sbi}	=	Allowable steel bearing stress under P_{jack} between wedge plate and bearing plate
f_{sy}	=	Yield strength of bearing plate or wedge plate material whichever is lower
n	=	Largest distance from outer edge of wedge plate to outer edge of bearing plate
ρ_s	=	Orthogonal reinforcement ratio in each of directions (vertical and horizontal) expressed as a percentage of distribution area
t	=	Thickness of bearing plate

5. The relationship between gross bearing plate area and distribution area shall satisfy the following conditions in the x and y direction:

   If $e_{min} > 0.5\ b$ then $a = 2b$

   If $e_{min} < 0.5\ b$ then $a = (b + 2\ e_{min})$

   but $e_{max} <\ 4\ e_{min}$

6. For transverse post-tensioning of roadway slabs, the bearing stress shall not exceed 0.9f'c at P_{jack} of all strands (before seating) or 4,000 psi at service load after all losses.

6-02.3(26)D Non-Bearing Type Anchorages

All anchorages that do not conform to Section 6-02.3(26)C shall be defined as non-bearing type anchorages. Except as allowed by Section 6-02.3(26)B, anchorages and post-tensioning systems with non-bearing type anchorages shall be qualified by test.

Anchorage Qualification Test

A minimum of three successful anchorage qualification tests are required for each tendon size. The materials for each qualification test shall be taken from different heats.

Test Block

The test block shall be a square or rectangular prism, depending on the shape of bearing plate. The test block shall conform to the following:

1. The test block width and depth in each direction shall be three inches plus the smaller of the following:
    a. Two times the minimum edge distance from the center of the bearing plate to the face of concrete.
    b. The minimum center-to-center spacing of the bearing plate
2. The length of a test block containing a single anchorage and local zone, loaded in a single machine, shall be at least two times the larger cross-sectional test block dimension.
3. The length of test block with an anchorage and local zone on either end, loaded by stressing a test tendon, shall be at least four times the larger cross-sectional test block dimension.
4. The first or local zone of reinforcement in the test block behind the anchorage for a distance equal to the largest of the two cross-sectional dimensions of the anchorage shall simulate the actual first or local zone of reinforcement used in the structure. For the remaining length of the test block, the reinforcement may be increased as required to prevent failure in that portion.
5. The concrete strength at the time of testing shall not exceed either the minimum strength specified for the system at the time of tensioning, nor 85 percent of the 28-day cylinder strength for normal weight concrete or 70 percent of the 28-day cylinder strength for lightweight concrete.

Test Procedure

The test force shall be applied to the wedge plate, or anchor nut, either in a testing apparatus or through an oversized tendon. The force shall be applied in stages to 40 percent and then to 80 percent of MUTS. At 40 percent MUTS, the force shall be held for 10 minutes to allow inspection for cracks. At 80 percent MUTS, the force shall be held for one hour. Thereafter the force shall be increased to at least 120 percent MUTS, and then either to failure or to the limit of testing equipment.

Acceptance Criteria

For forces up to 40 percent MUTS, the width of concrete cracks shall not exceed 0.002-inch.

After holding the force at 80 percent MUTS for one hour, the width of concrete cracks shall not exceed 0.01-inch.

The test block shall not fail prior to reaching 120 percent MUTS.

Post-Tensioning System Qualification Test

A minimum of one successful system qualification test for each tendon size is required for a representative full size tendon embedded in a concrete test block. The test shall establish that all tendon components, including the spiral, orthogonal, and surface steel reinforcing bars in the local zone, perform as required.

The test block shall conform to the requirements specified above for the anchorage qualification test.

The test procedure shall conform to the requirements specified above for the anchorage qualification test, except as noted. After the test force has been held at 80 percent MUTS for one hour, the force shall be increased to at least 95 percent MUTS.

The acceptance criteria shall be as specified above for the anchorage qualification test.

Wedge Plate Qualification Test

Wedge plates shall meet the following requirements. A minimum of three successful wedge plate tests, each from a different heat, for each tendon size are required:

1. After loading to 95 percent MUTS for the tendon and subsequent force release, the permanent deflection of the wedge plate's top surface shall not exceed $^1/_{600}$ of clear span. The load test shall be performed with the wedge plate support simulating conditions in the anchorage assembly. The force shall be applied by pulling on a sample tendon using the strand system wedges.
2. The wedge plate shall be tested to static load tests, or to the loading capacity of the testing equipment. The tests shall simulate actual tendon forces applied to the wedges. The failure force shall be at least 120 percent MUTS for the tendon.

6-02.3(26)E Ducts

Ducts shall be round, except that ducts for transverse post-tensioning of bridge deck slabs may be rectangular. Ducts shall conform to the following requirements for internal embedded installation and external exposed installation. Elliptical shaped duct may be used if approved by the Engineer.

Ducts for Internal Embedded Installation

For longitudinal tendons, the Contractor shall encase each tendon in a semi-rigid, galvanized, ferrous metal duct. Semi-rigid ducts shall be corrugated, and their minimum wall thickness shall be either 26 gage for ducts less than or equal to $2^5/_8$-inches in diameter, or 24 gage for ducts greater than $2^5/_8$-inches in diameter. For prestressing steel bars preassembled with their ducts, the minimum duct thickness shall be 31 gage. For transverse tendons, the Contractor shall encase each tendon in a rigid plastic duct. This duct shall maintain the required profile within a placement tolerance of plus or minus $^1/_4$-inch for longitudinal tendons and plus or minus $^1/_8$-inch for transverse slab tendons during all phases of the work. The ducts shall be completely sealed to keep out all mortar.

Each duct shall be located to place the tendon at the center of gravity alignment shown in the Plans. To keep friction losses to a minimum, the Contractor shall install ducts to the exact lines and grades shown in the Plans. Once in place, the ducts shall be tied firmly in position before they are covered with concrete. During concrete placement, the Contractor shall not displace or damage the ducts.

The ends of the ducts shall:

1. Permit free movement of anchorage devices, and
2. Remain covered after installation in the forms to keep out all water or debris.

The Contractor shall install vents at high points and drains at low points of the tendon profile (and at other places if the Plans require). Vents and drains shall be $^1/_2$-inch minimum diameter standard steel or polyethylene pipe. Vents shall point upward and remain closed until grouting begins. Drains shall point downward and remain open until grouting begins. Ends of steel vents and drains shall be removed 1-inch inside the concrete surface after grouting has been completed; polyethylene vents and drains may be left flush to the surface unless otherwise directed by the Engineer. Conduit vents are not required for transverse post-tensioning ducts in the roadway slab unless specified in the Plans.

Immediately after any concrete placement, the Contractor shall force blasts of oil-free, compressed air through the ducts to break up and remove any mortar inside before it hardens. Before deck concrete is placed, the Contractor shall satisfy the Engineer that ducts are unobstructed and contain nothing that could interfere with tendon installation, tensioning, or grouting. If the tendons are in place, the Contractor shall show that they are free in the duct.

In temperatures below 32ºF, ducts shall be kept free from water to avoid damage from freezing.

Strand tendon duct shall have an inside cross-sectional area large enough to accomplish strand installation and grouting. The area of the duct shall be at least 2.5 times the net area of prestressing steel in the duct. The maximum duct diameter shall be 4$^1/_2$-inches.

The inside diameter of bar tendon duct shall at least be $^1/_4$-inch larger than the bar diameter. At coupler locations the duct diameter shall at least be $^1/_4$-inch larger than the coupler diameter.

Ducts installed and cast into concrete prior to prestressing steel installation, shall be capable of withstanding at least 10-feet of concrete fluid pressure.

Ducts shall have adequate longitudinal bending stiffness for smooth, wobble free placement. A minimum of three successful duct qualification tests are required for each diameter and type of duct, as follows:

1. Ducts with diameters 2-inches and smaller shall not deflect more than 3-inches under its own weight, when a 10 foot. duct segment is supported at its ends.
2. Ducts larger than 2-inches in diameter shall not deflect more than 3-inches under its own weight, when a 20 foot duct segment is supported at its ends.
3. Duct shall not dent more than $^1/_8$-inch under a concentrated load of 100 pounds applied between corrugations by a #4 steel reinforcing bar.

When the duct must be bent in a tight radius, more flexible duct may be used, subject to the Engineer’s approval.

Ducts for External Exposed Installation

Duct shall be high-density polyethylene (HDPE) conforming to ASTM D 3350. The cell classification for each property listed in Table 1 shall be as follow:

Property	Cell Classification
1	3 or 4
2	2, 3, or 4
3	4 or 5
4	4 or 5
5	2 or 3
6	2, 3, or 4

The color code shall be C.

Duct for external tendons, including their splices, shall be water tight, seamless or welded, and be capable of resisting at least 150-psi grout pressure.

Transition couplers between ducts shall conform to either the standard pressure ratings of ASTM D 3505 or the hydrostatic design stresses of ASTM F 714 at 73°F. The inside diameter through the coupled length shall not be less than that produced by the dimensional tolerances specified in ASTM D 3505.

Workers performing HDPE pipe welding shall have satisfactorily completed a certified HDPE pipe welding course and shall have a minimum of five years experience in welding HDPE pipe.

The Contractor shall submit the name and HDPE pipe welding work experience of each HDPE pipe welder proposed to perform this work in the project. The experience submittal for each HDPE pipe welder shall include:

The Engineer may require the HDPE pipe welder to demonstrate test HDPE pipe welding before receiving final approval.

1. The name of the pipe welder.
2. The name, date, and location of the certified HDPE pipe welding course, with the course completion certificate.
3. A list of at least three projects in the last five years where the pipe welder performed HDPE pipe welding, including:
    a. The project name and location, and date of construction.
    b. The Governmental Agency/Owner.
    c. The name, address, and phone number of the Governmental Agency/Owner's representative.

The Contractor shall not begin HDPE pipe welding operations until receiving the Engineer's approval of the work experience submittal for each HDPE pipe welder performing HDPE pipe welding in the project.

Transitions

Transitions between ducts and wedge plates shall have adequate length to reduce the angle change effect on the performance of strand-wedge connection, friction loss at the anchorage, and fatigue strength of the post-tensioning reinforcement.

6-02.3(26)F Prestressing Reinforcement

All prestressing reinforcement strand shall comply with Section 9-07.10. They shall not be coupled or spliced. Tendon locations shown in the Plans indicate final positions after stressing (unless the Plans say otherwise). No tendon made of 7 wire strands shall contain more than 37 strands of $^1/_2$-inch diameter, or more than 27 strands of 0.6-inch diameter.

All prestressing reinforcement bar shall conform to Section 9-07.11. They shall not be coupled or spliced except as otherwise specified in the Plans or Special Provisions.

Prestressing reinforcement not conforming to either Section 9-07.10 or 9-07.11 will not be allowed except as otherwise noted. Such reinforcement may be used provided it is specifically allowed by the Plans or Special Provisions, it satisfies all material and performance criteria specified in the Plans or Special Provisions, and receives the Engineer's approval.

From the time prestressing reinforcement is manufactured until it is grouted or encased in concrete, the Contractor shall protect it from dirt, grease, rust, corrosives, and all physical damage. The Engineer will reject prestressing reinforcement that shows any sign of damage, rust, or corrosion. If the prestressing reinforcement will not be stressed and grouted for more that ten calendar days after it is placed in the ducts, the Contractor shall place an approved corrosion inhibitor in the ducts.

The feeding ends of the strand tendons shall be equipped with a bullet nosing or similar apparatus to facilitate strand tendon installation.

Strand tendons may be installed by pulling or pushing. Any equipment capable to performing the task may be used, provided it does not damage the strands and conforms to the following:

1. Pulling lines shall have a capacity of at least 2.5 times the dead weight of the tendons when used for essentially horizontal tendon installation.
2. Metal pushing wheels shall not be used.
3. Bullets for checking duct clearance prior to concreting shall be rigid and be $^1/_8$-inch smaller than the inside diameter of the duct. Bullets for checking duct after concreting shall be less than $^1/_4$-inch smaller than the inside diameter of the duct.

6-02.3(26)G Tensioning

Equipment for tensioning post-tensioning reinforcement shall meet the following requirements:

1. Stressing equipment shall be capable to produce a jacking force of at least 80 percent MUTS of the post-tensioning reinforcement.
2. Jacking force test capacity shall be at least 95 percent MUTS of the post-tensioning reinforcement.
3. Wedge seating methods shall assure uniform seating of wedge segments and uniform wedge seating losses on all strand tendons.
4. Accumulation of differential seating losses during tensioning cycling shall be prevented by proper devices.
5. Jacks used for stressing tendons less than 20-feet long shall have wedge power seating capability.

The Contractor shall not begin to tension the tendons until:

1. All concrete has reached a compressive strength of at least 4,000 psi or the strength specified in the Plans (demonstrated on test cylinders made of the same concrete cured under the same conditions as that in the bridge), and
2. The Engineer is satisfied that all strands are free in the ducts.

Tendons shall be tensioned to the values shown in the Plans (or approved shop drawings) with hydraulic jacks. When stressing from both ends of a tendon is specified, it need not be simultaneous unless otherwise specified in the Plans. The jacking sequence shall follow the approved shop drawings.

Each jack shall have a pressure gauge that will determine the load applied to the tendon. The gauge shall display pressure accurately and readably with a dial at least 6-inches in diameter or with a digital display. Each jack and its gauge shall be calibrated as a unit and shall be accompanied by a certified calibration chart. The Contractor shall provide one copy of this chart to the Engineer for use in monitoring. The cylinder extension during calibration shall be in approximately the position it will occupy at final jacking force.

All jacks and gauges must be recalibrated and recertified: (1) at least every 180 days, and (2) after any repair or adjustment. The Engineer may use pressure cells to check jacks, gauges, and calibration charts before and during tensioning.

These stress limits apply to all tendons (unless the Plans set other limits):

1. Maximum service load after all losses: 80 percent of the specified yield point stress of the steel.
2. Maximum tensile stress during jacking: 80 percent MUTS of the tendon.
3. Maximum initial stress at anchorage after seating: 70 percent MUTS of the tendon.

Tendons shall be anchored at initial stresses that will ultimately maintain service loads at least as great as the Plans require.

As stated in Section 6-02.3(26)A, the assumed design friction coefficient "μ" and wobble coefficient "k" shown in the Plans shall be used to calculate the stressing elongation. These coefficients may be revised by the post-tensioning supplier by the following method provided it is approved by the Engineer:

> Early in the project, the post-tensioning supplier shall test, in place, two representative tendons of each size and type shown in the Plans, for the purpose of accurately determining the friction loss in a strand and/or bar tendon.
>
> The test procedure shall consist of stressing the tendon at an anchor assembly with load cells at the dead end and jacking end. The test specimen shall be tensioned to 80 percent of ultimate in ten increments. For each increment, the gauge pressure, elongation, and load cell force shall be recorded and the data furnished to the Engineer. The theoretical elongations and post-tensioning forces shown on the post-tensioning shop drawings shall be re-evaluated by the post-tensioning supplier using the results of the tests and corrected as necessary. Revisions to the theoretical elongations shall be submitted to the Engineer for evaluation and approval. The apparatus and methods used to perform the tests shall be proposed by the post-tensioning supplier and be subject to the approval of the Engineer.
>
> All costs associated with testing and evaluating test data shall be included in the unit contract prices for the applicable items of work involved.

As tensioning proceeds, the Engineer will be recording the applied load, tendon elongation, and anchorage seating values.

Elongation measurements shall be made at each stressing location to verify that the tendon force has been properly achieved. If proper anchor set has been achieved and the measured elongation of each strand tendon is within plus or minus 7 percent of the approved calculated elongation, the stressed tendon represented by the elongation measurements is acceptable to the Contracting Agency.

In the event discrepancies greater than 7 percent exist between the measured and calculated elongations, the jack calibration shall be checked and stressing records reviewed for any evidence of wire or strand breakage. If the jack if properly calibrated and there is no evidence of wire or strand breakage, a force verification lift off shall be performed to verify the force in the tendon. The post-tensioning supplier force verification lift off procedure shall provide access for visual verification of anchor plate lift off. The jacking equipment shall be capable of bridging and lifting off the anchor plate. The tendon is acceptable if the verification lift off force is not less than 99 percent of the approved calculated force nor more than 70 percent of the specified minimum ultimate tensile strength of the prestressing steel or as approved by the Engineer.

Elongation measurements shall be recorded for bar tendons to verify proper tensioning only. Acceptance will be by force verification lift off. The bar tendon is acceptable if the verification lift off force is not less than 95 percent nor more than 105 percent of the approved calculated force or as approved by the Engineer.

When removing the jacks, the Contractor shall relieve stresses gradually before cutting the prestressing reinforcement. The prestressing strands shall be cut a minimum of 1-inch from the face of the anchorage device.

6-02.3(26)H Grouting

After tensioning the tendons, the Contractor shall again blow oil-free, compressed air through each duct. All drains shall then be closed and the vents opened. Grout caps shall be installed at tendon ends prior to grouting. After completely filling the duct with grout, the Contractor shall pump the grout from the low end at a pressure of not more than 250 psig, except for transverse tendons in deck slabs the grout pressure shall not exceed 100 psig. Grout shall be continuously wasted through each vent until no more air or water pockets show. At this point, all vents shall be closed and grouting pressure at the injector held between 100 and 200 psig for at least 10 seconds, except for transverse tendons in deck slabs the grouting pressure shall be held between 50 and 75 psig for at least 10 seconds. The Contractor shall leave all plugs, caps, and valves in place and closed for at least 24 hours after grouting.

Grouting equipment shall:

1. Include a pressure gauge with an upper end readout of between 275 and 325 psig;
2. Screen the grout before it enters the pump with an easily reached screen that has clear openings of no more than 0.125-inches;
3. Be gravity fed from an attached, overhead hopper kept partly full during pumping; and
4. Be able to complete the largest tendon on the project in no more than 20 minutes of continuous grouting.

In addition, the Contractor shall have standby equipment (with a separate power source) available for flushing the grout when the regular equipment cannot maintain a one-way flow of grout. This standby equipment shall be able to pump at 250 psig.

The grout shall consist of Portland cement, water, and a water reducing admixture and shall be mixed in the following proportions:

Portland Cement Type I or II	1 Sack
Water	4.5 Gallons Maximum
Water Reducing Admixture	Manufacturer's Recommendation
Fly Ash (Optional)	20 Pounds Maximum

The water reducing admixture shall be limited to AASHTO M 194 Type A or D and shall not contain ingredients that may corrode steel (that is chlorides, fluorides, sulfates, or nitrates). Fly ash may be used at the option of the Contractor.

The Contractor shall proportion the mix to produce a grout with a flow of 11 to 20 seconds as determined by WSDOT Test Method for ASTM C 939, Flow of Grout for Preplaced Aggregate Concrete (Flow Cone Method).

The grout ejected from the end vent shall have a minimum flow of 11 seconds.

The grout mix shall be injected within 30 minutes after the water is added to the cement. Temperature of the surrounding concrete shall be at least 35ºF from the time the grout injecting begins until 2-inch cubes of the grout have a compressive strength of 800 psi. Cubes shall be made in accordance with WSDOT Test Method T 813 and stored in accordance with WSDOT FOP for AASHTO T 23. If ambient conditions are such that the surrounding concrete temperature may fall below 35ºF, the Contractor shall provide a heat source and protective covering for the structure to keep the temperature of the surrounding concrete above 35ºF. Grout temperature shall not exceed 90ºF during mixing and pumping. If conditions are such that the temperature of the grout mix may exceed 90ºF, the Contractor will make necessary provisions, such as cooling the mix water and/or dry ingredients, to ensure that the temperature of the grout mix does not exceed 90ºF.

6-02.3(27) Concrete for Precast Units

Precast units shall not be removed from forms until the concrete has attained a minimum compressive strength of 70 percent of the specified design strength as verified by rebound number determined in accordance with WSDOT FOP for ASTM C 805.

Precast units shall not be shipped until the concrete has reached the specified design strength as determined by testing cylinders made from the same concrete as the precast units. The cylinders shall be made, handled, and stored in accordance with WSDOT FOP for AASHTO T 23 and compression tested in accordance with AASHTO Test Method T 22 and AASHTO Test Method T 231.

Self compacting concrete (SCC) may be used for precast concrete barrier covered under Section 6-10 and drainage items covered under Section 9-12. If self compacting concrete has been approved for use the requirements of Section 6-02.3(4)C consistency shall not apply. Self compacting concrete is concrete that is able to flow under its own weight and completely fill the formwork, even in the presence of dense reinforcement, without the need of any vibration, while maintaining homogeneity. When using SCC modified testing procedures for air content and compressive strength will be used. The modification shall be that molds will be filled completely in one continuous lift without any rodding, vibration, tamping or other consolidation methods other than lightly taping around the exterior of the mold with a rubber mallet to allow entrapped air bubbles to escape. In addition the fabricators QC testing shall include Slump Flow Test results, which do not indicate segregation. As part of the plants approval for use of SCC the plant fabricator shall cast one barrier, or drainage item and have that barrier or drainage item sawed in half for examination by the Contracting Agency to determine that segregation has not occurred.

6-02.3(28) Precast Concrete Panels

The Contractor shall perform quality control inspection. The manufacturing plant for precast concrete units shall be certified by the Precast/Prestressed Concrete Institute's Plant Certification Program for the type of precast member to be produced, or the National Precast Concrete Association's Plant Certification Program or be an International Congress Building Officials or International Code Council Evaluation Services recognized fabricator of structural precast concrete products, and shall be approved by WSDOT as a Certified Precast Concrete Fabricator prior to the start of production. WSDOT Certification will be granted at, and renewed during, the annual precast plant review and approval process. Products that shall conform to this requirement include noise barrier panels, wall panels, floor and roof panels, marine pier deck panels, retaining walls, pier caps, and bridge deck panels. Precast concrete units that are prestressed shall meet all the requirements of Section 6-02.3(25).

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the work and verify the quality of that work. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and workmanship.

Prior to the start of production of the precast concrete units, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the work. If the Inspector observes any nonspecification work or unacceptable quality control practices, the Inspector will advise the plant manager. If the corrective action is not acceptable to the Engineer, the unit(s) will be rejected.

6-02.3(28)A Shop Drawings

Before casting the structural elements, the Contractor shall submit:

1. Seven sets of shop drawings for approval by the Bridge and Structures Engineer, Department of Transportation, Transportation Building, Olympia, WA 98504; and
2. Two sets of shop drawings to the Project Engineer.

These shop drawings shall show complete details of the methods, materials, and equipment the Contractor proposes to use in prestressing/precasting work. The shop drawings shall follow the design conditions shown in the Plans unless the Engineer approves equally effective variations.

The shop drawings shall contain as a minimum:

1. Unit shapes (elevations and sections) and dimensions.
2. Finishes and method of constructing the finish (i.e., forming, rolling, etc.).
3. Reinforcing, joint, and connection details.
4. Lifting, bracing, and erection inserts.
5. Locations and details of hardware attached to the structure.
6. Relationship to adjacent material.

Approval of these shop drawings shall not relieve the Contractor of responsibility for accuracy of the drawings or conformity with the Contract. Approval will not indicate a check on dimensions.

The Contractor may deviate from the approved shop drawings only after obtaining the Engineer's approval of a written request that describes the proposed changes. Approval of a change in method, material, or equipment shall not relieve the Contractor of any responsibility for completing the work successfully.

Before completion of the Contract, the Contractor shall provide the Engineer with reproducible originals of the shop drawings (and any approved changes). These shall be clear, suitable for microfilming, and on permanent sheets that conform with the size requirements of Section 6-01.9.

6-02.3(28)B Casting

Before casting precast concrete units, the Contractor and Fabrication Inspector shall have possession of an approved set of shop drawings.

Concrete shall meet requirements of Section 6-02.3(25)B for annual pre-approval of the concrete mix design, and slump.

Precast units shall not be removed from forms until the concrete has attained a minimum compressive strength of 70 percent of the specified design strength. A minimum compressive strength at other than 70 percent may be used for specific precast units if the fabricator requests and receives approval as part of the WSDOT plant certification process.

Forms may be steel or plywood faced, providing they impart the required finish to the concrete.

6-02.3(28)C Curing

Concrete in the precast units shall be cured by either moist or accelerated curing methods. The methods to be used shall be preapproved in the WSDOT plant certification process.

1. For moist curing, the surface of the concrete shall be kept covered or moist until such time as the compressive strength of the concrete reaches the strength specified for stripping. Exposed surfaces shall be kept continually moist by fogging, spraying, or covering with moist burlap or cotton mats. Moist curing shall commence as soon as possible following completion of surface finishing.
2. For accelerated curing, heat shall be applied at a controlled rate following the initial set of concrete in combination with an effective method of supplying or retaining moisture. Moisture may be applied by a cover of moist burlap, cotton matting, or other effective means. Moisture may be retained by covering the unit with an impermeable sheet.

Heat may be radiant, convection, conducted steam or hot air. Heat the concrete to no more than 100ºF during the first two hours after pouring the concrete, and then increase no more than 25ºF per hour to a maximum of 175ºF. After curing is complete, cool the concrete no more than 25ºF per hour to 100ºF. Maintain the concrete temperature above 60ºF until the unit reaches stripping strength.

Concrete temperature shall be monitored by means of a thermocouple embedded in the concrete (linked with a thermometer accurate to plus or minus 5ºF). The recording sensor (accurate to plus or minus 5ºF) shall be arranged and calibrated to continuously record, date, and identify concrete temperature throughout the heating cycle. This temperature record shall be made available to the Engineer for inspection and become a part of the documentation required.

The Contractor shall never allow dry heat to directly touch exposed unit surfaces at any point.

6-02.3(28)D Contractors Control Strength

The concrete strength at stripping and the verification of design strength shall be determined by testing cylinders made from the same concrete as the precast units. The cylinders shall be made, handled, and stored in accordance with WSDOT FOP for AASHTO T 23 and compression tested in accordance with AASHTO Test Method T 22 and AASHTO Test Method T 231.

For accelerated cured units, concrete strength shall be measured on test cylinders cast from the same concrete as that in the unit. These cylinders shall be cured under time-temperature relationships and conditions that simulate those of the unit. If the forms are heated by steam or hot air, test cylinders will remain in the coolest zone throughout curing. If forms are heated another way, the Contractor shall provide a record of the curing time-temperature relationship for the cylinders for each unit to the Engineer. When two or more units are cast in a continuous line and in a continuous pour, a single set of test cylinders may represent all units provided the Contractor demonstrates uniformity of casting and curing to the satisfaction of the Engineer.

The Contractor shall mold, cure, and test enough of these cylinders to satisfy specification requirements for measuring concrete strength. The Contractor may use 4-inch by 8-inch or 6-inch by 12-inch cylinders. The Contractor shall let cylinders cool for at least one-half hour before testing for release strength.

Test cylinders may be cured in a moist room or water tank in accordance with WSDOT FOP for AASHTO T-23 after the unit concrete has obtained the required release strength. If, however, the Contractor intends to ship the unit prior to standard 28-day strength test, the design strength for shipping shall be determined from cylinders placed with the unit and cured under the same conditions as the unit. These cylinders may be placed in a noninsulated, moisture-proof envelope.

To measure concrete strength in the precast unit, the Contractor shall randomly select two test cylinders and average their compressive strengths. The compressive strength in either cylinder shall not fall more than 5 percent below the specified strength. If these two cylinders do not pass the test, two other cylinders shall be selected and tested.

6-02.3(28)E Finishing

The Contractor shall provide a finish on all relevant concrete surfaces as defined in Section 6-02.3(14), unless the Plans or Special Provisions require otherwise.

6-02.3(28)F Tolerances

The units shall be fabricated as shown in the Plans, and shall meet the dimensional tolerances listed in PCI MNL-116-85, unless otherwise required by the Plans or Special Provisions.

6-02.3(28)G Handling and Storage

The Contractor shall lift all units only by adequate devices at locations designated on the shop drawings. When these devices and locations are not shown in the Plans, Section 6-02.3(25)L shall apply.

Precast units shall be stored off the ground on foundations suitable to prevent differential settlement or twisting of the units. Stacked units shall be separated and supported by dunnage of uniform thickness capable of supporting the units. Dunnage shall be arranged in vertical planes. The upper units of a stacked tier shall not be used as storage areas for shorter units unless substantiated by engineering analysis and approved by the Engineer.

6-02.3(28)H Shipping

Precast units shall not be shipped until the concrete has reached the specified design strength, and the Engineer has reviewed the fabrication documentation for contract compliance and stamped the precast concrete units " Approved for Shipment". The units shall be supported in such a manner that they will not be damaged by anticipated impact on their dead load. Sufficient padding material shall be provided between tie chains and cables to prevent chipping or spalling of the concrete.

6-02.3(28)I Erection

When the precast units arrive on the project, the Project Engineer will confirm that they are stamped "Approved for Shipment." The Project Engineer will evaluate the present units for damage before accepting them.

The Contractor shall lift all units by suitable devices at locations designated on the shop drawings. Temporary shoring or bracing shall be provided, if necessary. Units shall be properly aligned and leveled as required by the Plans. Variations between adjacent units shall be leveled out by a method approved by the Engineer.

6-02.4 Measurement

Except as noted below, all classes of concrete shall be measured in place by the cubic yard to the neat lines of the structure as shown in the Plans.

Exception: concrete in cofferdam seals. Payment for Class 4000W concrete used in these seals will be based on the volume calculated using the neatline dimensions for the seal as shown in the contract plans. For calculated purposes, the horizontal dimension will be increased by 1 foot outside the seal neatline perimeter. The vertical dimension is the distance between the top and bottom neatline elevations. No payment will be made for any concrete that lies outside of these limits to accommodate the Contractor's cofferdam configuration. If the Engineer eliminates the seal in its entirety a contract change order will be issued.

Exception: concrete in a separate lump-sum, superstructure bid item. Any concrete quantities noted under this item in the Special Provisions will not be measured. Although the Special Provisions list approximate quantities for the Contractor's convenience, the Contracting Agency does not guarantee the accuracy of these estimates. Before submitting a bid, the Contractor shall have verified the quantities. Even though actual quantities used may vary from those listed in the Special Provisions, the Contracting Agency will not adjust the lump sum contract price for superstructure (except for approved changes).

The Contracting Agency will pay for no concrete placed below the established elevation of the bottom of any footing or seal.

Lean concrete will be measured by the cubic yard for the quantity of material placed per the producer's invoice, except that lean concrete included in other contract items will not be measured.

No deduction will be made for pile heads, reinforcing steel, structural steel, bolts, weep holes, rustications, chamfers, edgers, joint filler, junction boxes, miscellaneous hardware, ducts or less than 6-inch diameter drain pipes when computing concrete quantities for payment.

All reinforcing steel will be measured by the computed weight of all metal actually in place and required by the Plans or the Engineer. Epoxy-coated bars will be measured before coating. The Contractor shall furnish (without extra allowance):

1. Spreaders, form blocks, wire clips, and other fasteners.
2. Extra steel in splices not shown in the Plans.
3. Extra shear steel at construction joints not shown in the Plans when the Engineer permits such joints for the Contractor's convenience.

The following table shall be used to compute weight of reinforcing steel:

Steel Reinforcing Bar

Deformed Bar Designation Number	Nominal Diameter Inches	Unit Weight Pounds per Foot
3	0.375	0.376
4	0.500	0.668
5	0.625	1.043
6	0.750	1.502
7	0.875	2.044
8	1.000	2.670
9	1.128	3.400
10	1.270	4.303
11	1.410	5.313
14	1.690	7.650
18	2.260	13.600

Gravel backfill will be measured as specified in Section 2-09.4.

6-02.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

"Conc. Class ____", per cubic yard.

"Commercial Concrete", per cubic yard.

All concrete, except in Superstructure when this is covered by a separate bid item, will be paid for at the unit contract price per cubic yard in place for the various classes of concrete.

"Superstructure (name bridge)", lump sum.

All costs in connection with providing holes for vents, for furnishing and installing cell drainage pipes for box girder structures, and furnishing and placing grout and shims under steel shoes shall be included in the unit contract prices for the various bid items involved.

All costs in connection with the construction of weep holes, including the gravel backfill for drains surrounding the weep holes except as provided in Section 2-09.4, shall be included by the Contractor in the unit contract price per cubic yard for "Conc. Class ____".

"Lean Concrete", per cubic yard.

Lean concrete, except when included in another bid item, will be paid for at the unit contract price per cubic yard.

"St. Reinf. Bar", per pound.

"Epoxy-Coated St. Reinf. Bar", per pound.

Payment for reinforcing steel shall include the cost of furnishing, fabricating, and placing the reinforcement. In structures of reinforced concrete where there are no structural steel bid items, such minor metal parts as expansion joints, bearing assemblies, and bolts will be paid for at the unit contract price for "Reinforcing Bar" unless otherwise specified.

"Gravel Backfill for Foundation Class A", per cubic yard.

"Gravel Backfill for Foundation Class B", per cubic yard.

"Gravel Backfill for Wall", per cubic yard.

"Deficient Strength Conc. Price Adjustment", by calculation.

"Deficient Strength Conc. Price Adjustment" will be calculated and paid for as described in Section 6-02.3(5)L. For the purpose of providing a common proposal for all bidders, the Contracting Agency has entered an amount for the item "Deficient Strength Conc. Price Adjustment" in the bid proposal to become a part of the total bid by the Contractor. The item "Deficient Strength Conc. Price Adjustment" covers all applicable classes of concrete.

6-03 STEEL STRUCTURES

6-03.1 Description

This work consists of furnishing, fabricating, erecting, cleaning, and painting steel structures and the structural steel parts of nonsteel structures

6-03.2 Materials

Materials shall meet the requirements of the following sections:

Structural Steel and Related Materials	9-06
Paints	9-08

Structural steel shall be classified as:

1. Structural carbon steel (to be used whenever the Plans do not specify another classification),
2. Structural low alloy steel, and
3. Structural high strength steel.

Unless the Plans or Special Provisions state otherwise, the following shall be classified as structural carbon steel: shims; ladders; stairways; anchor bolts and sleeves; pipe, fittings and fastenings used in handrails; and other metal parts, even if made of other materials, for which payment is not specified.

All AASHTO M 270 material used in what the Plans show as main load-carrying tension members or as tension components of flexural members shall meet the Charpy V-notch requirements of AASHTO M 270 temperature zone 2. All AASHTO M 270 material used in what the Plans show as fracture critical members shall meet the Charpy V-notch requirements of AASHTO M 270, Fracture Critical Impact Test Requirements, temperature zone 2. Charpy V-notch requirements for other steel materials shall be as specified in the Plans and Special Provisions.

The Contractor shall submit for the Engineer's approval a written plan for visibly marking the material so that it can be traced. These marks shall remain visible at least through the fit-up of the main load-carrying tension members. The marking method shall permit the Engineer to verify: (1) material specification designation, (2) heat number, and (3) material test reports to meet any special requirements.

For steel in main load-carrying tension members and in tension components of flexural members, the Contractor shall include the heat numbers on the reproducible copies of the as-built shop plans.

6-03.3 Construction Requirements

Structural steel fabricators of girders, floorbeams, truss members, and stringers, for permanent steel bridges, shall be certified under the AISC Quality Certification Program, Major Steel Bridges Category. When fracture critical members are specified in the contract, structural steel fabricators shall also have an endorsement F, Fracture Critical, under the AISC Quality Certification Program.

6-03.3(1) Vacant

6-03.3(2) Facilities for Inspection

The Contractor shall provide all facilities the Inspector requires to inspect material and workmanship. Inspectors shall be given safe and free access to all areas in the mill and shop.

6-03.3(3) Inspector's Authority

The Inspector may reject materials or workmanship that does not comply with these Specifications. In any dispute, the Contractor may appeal to the Engineer whose decision shall be final.

By its inspection at the mill and shop, the Contracting Agency intends only to facilitate the work and prevent errors. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material or workmanship.

6-03.3(4) Rejections

Even if the Inspector accepts materials or finished members, the Contracting Agency may later reject them if defective. The Contractor shall promptly replace or make good any rejected materials or workmanship.

6-03.3(5) Mill Orders and Shipping Statements

The Contractor shall furnish as many copies of mill orders and shipping statements as the Engineer requires.

6-03.3(6) Weighing

Structural steel need not be weighed unless the Plans or Special Provisions require it. When a weight is required, it may either be calculated or obtained by scales. The Contractor shall furnish as many copies of the calculations or weight slips as the Engineer requires. If scale weights are used, the Contractor shall record separately the weights of all tools, erection material, and dunnage.

6-03.3(7) Shop Plans

The Contractor shall submit for approval all shop detail plans for fabricating the steel. These shall be sent to the Bridge and Structures Engineer, Department of Transportation, Transportation Building, Olympia, WA 98504. If these plans will be submitted directly from the fabricator, the Contractor shall so notify the Project Engineer in writing.

Prints of the plans shall be supplied in these quantities:

1. Eight sets to the Bridge and Structures Engineer (four more sets are required for each affected railroad company on any grade separation structure that carries a railroad over a highway), and
2. Two sets to the Project Engineer.

The Bridge and Structures Engineer will return the plans to the Project Engineer, who will forward copies to the Contractor. If any sheets require correction, the Contractor shall correct and resubmit them in the quantities required above. No material shall be fabricated until: (1) the Bridge and Structures Engineer has approved the plans, and (2) the State Materials Engineer has approved the materials source and the fabricator.

In approving shop plans, the Contracting Agency accepts only the nature and scope of the details without validating any dimensions.

Unless the Engineer permits it in writing, no changes shall be made in any drawing after its approval.

Before physical completion of the project, the Contractor shall furnish the Project Engineer one set of reproducible copies of the as-built shop plans. (One more set is required for each affected railroad company on any grade separation structure that carries a railroad over a highway.) The reproducible copies shall be clear, suitable for microfilming, and on permanent sheets that measure no smaller than 11 by 17-inches. Alternatively, the shop drawings may be provided in an electronic format with the approval of the Bridge and Structures Engineer.

6-03.3(7)A Erection Methods

Before beginning to erect any steel structure, the Contractor shall submit to the Engineer for review and shall have received approval for the erection plan and procedure describing the methods the Contractor intends to use. The Contractor's erection plan and procedure shall be reviewed by the steel fabricator prior to being submitted to the Engineer. The Contractor shall submit evidence that the fabricator has reviewed the erection plans and procedure; and submit the fabricator's review comments to the Engineer along with the erection plan submittal.

The erection plan and procedure shall provide complete details of the erection process including but not limited to:

1. Temporary falsework support, bracing, guys, deadmen, and attachments to other structure components or objects;
2. Procedure and sequence of operation;
3. Girder stresses during progressive stages of erection;
4. Girder masses , lift points, and lifting devices, spreaders, glommers, etc.;
5. Crane(s) make and model, mass, geometry, lift capacity, outrigger size and reactions;
6. Girder launcher or trolley details and capacity (if intended for use); and
7. Locations of cranes, barges, trucks delivering girders, and the location of cranes and outriggers relative to other structures, including retaining walls and wing walls.

The erection plan shall include drawings, notes, catalog cuts, and calculations clearly showing the above listed details, assumptions, and dimensions. Material properties, specifications, structural analysis, and any other data used shall also be included. The plan shall be prepared by (or under the direct supervision of) a Professional Engineer, licensed under Title 18 RCW, State of Washington, in the branch of Civil or Structural, and shall carry the engineer's seal and signature, in accordance with Section 6-02.3(16).

The Contractor shall submit the erection plans, calculations, procedure, and fabricator's comments directly to the Bridge and Structures Office, Construction Support Engineer, in accordance with Section 6-02.3(16). After the plan is approved and returned to the Contractor, all changes that the Contractor proposes shall be submitted to the Project Engineer for review and approval.

6-03.3(8) Substitutions

The Contractor shall not substitute sections that differ from plan dimensions unless the Engineer approves in writing. If the Contractor requests and receives approval to substitute heavier members, the Contracting Agency shall not pay any added cost.

6-03.3(9) Handling, Storing, and Shipping of Materials

Markings applied at the mill shall distinguish structural low alloy steel from structural carbon steel. The fabricator shall keep the two classes of steel carefully separated.

Before fabrication, all material stored at the fabricating plant shall be protected from rust, dirt, oil, and other foreign matter. The Contracting Agency will accept no rust-pitted material.

After fabrication, all material awaiting shipment shall be subject to the same storage requirements as unfabricated material.

All structural steel shall arrive at the job in good condition. As the Engineer requires, steel damaged by salt water shipment shall be thoroughly cleaned by high pressure water flushing, chemical cleaning, or sandblasting, and repainted with the specified shop coat.

All material shall be stored so as to prevent rust and loss of small parts. Piled material shall not rest on the ground or in water but on skids or platforms.

The loading, transporting, unloading, and piling of the structural steel material shall be so conducted that the metal will be kept clean and free from injury from rough handling.

In field assembly of structural parts, the Contractor shall use methods and equipment not likely to twist, bend, deform, or otherwise injure the metal. Any member slightly bent or twisted shall be corrected before it is placed. The Contracting Agency will reject any member with serious handling damage.

Girder sections shall be handled so as to prevent damage to the girders. If necessary, the Contractor shall provide temporary stiffeners to prevent buckling during erection.

6-03.3(10) Straightening Bent Material

If the Engineer permits in writing, plates, angles, other shapes, and built-up members may be straightened. Straightening methods shall not fracture or injure the metal. Distorted members shall be straightened mechanically. A limited amount of localized heat may be applied only if carefully planned and supervised, and only if the Engineer has approved a heat-straightening procedure in writing.

Parts to be heat-straightened shall be nearly free from all stress and external forces except those that result from the mechanical pressure used with the heat.

After straightening, the Contractor will inspect the member for fractures using a method determined by the Contracting Agency.

The Contracting Agency will reject metal showing sharp kinks and bends.

The procedure for heat straightening of universal mill (UM) plates by the mill or the fabricator shall be submitted to the Engineer for approval.

6-03.3(11) Workmanship and Finish

Workmanship and finish shall be first-class, equaling the best practice in modern bridge fabrication shops. Welding, shearing, burning, chipping, and grinding shall be done neatly and accurately. All parts of the work exposed to view shall be neatly finished.

Wherever the Plans show a surface finish symbol, the surface shall be machined.

6-03.3(12) Falsework

All falsework shall meet the requirements of Section 6-02.

6-03.3(13) Fabricating Tension Members

Plates for main load-carrying tension members or tension components of flexural members shall be:

1. Blast cleaned entirely or blast cleaned on all areas within 2-inches of welds to SSPC-SP6, Commercial Blast Cleaning, and
2. Fabricated from plate stock with the primary rolling direction of the stock parallel to the length of the member.

6-03.3(14) Edge Finishing

All rolled, sheared, and thermal cut edges shall be true to line and free of rough corners and projections. Corners along exposed edges shall be rounded to a minimum radius of 1/16-inch.

Sheared edges on plates more than 5/8-inch thick shall be planed, milled, ground, or thermal cut to a depth of at least 1/8-inch.

Re-entrant corners or cuts shall be filleted to a minimum radius of 3/4-inch.

Exposed edges of main load-carrying tension members or tension components of flexural members shall have a surface roughness no greater than 250 micro-inches as defined by the American National Standards Institute, ANSI B46.1, Surface Texture. Exposed edges of other members shall have surface roughness no greater than 1,000 micro-inches.

The Rockwell hardness of thermal-cut edges of structural low alloy or high-strength steel flanges, as specified in Section 9-06.2 and 9-06.3, for main load-carrying tension members or tension components of flexural members shall not exceed RHC 30. The fabricator shall prevent excessive hardening of flange edges through preheating, post heating, or control of the burning process as recommended by the steel manufacturer and approved by the Engineer.

Hardness testing shall consist of testing thermal-cut edges with an approved portable hardness tester. The hardness tester, and it's operating test procedures, shall be submitted to the Engineer for approval prior to use. The hardness tester shall be convertible to Rockwell C scale values.

At two locations, two tests shall be performed on each thermal-cut edge, one each within 1/4-inch of the top and bottom surfaces. The tests shall be located 1/4-the length of each thermal-cut edge from each end of the cut. If one or more readings are greater than RHC 30, the entire length of the edge shall be ground or machined to a depth sufficient to provide acceptable readings upon further retests. If thermal-cutting operations conform to procedures approved by the Engineer, and hardness testing results are consistently within acceptable limits, the Engineer may approve a reduction in the testing frequency.

6-03.3(15) Planing of Bearing Surfaces

Ends of columns that bear on base and cap plates shall be milled to true surfaces and accurate bevels.

When assembled, caps and base plates of columns and the sole plates of girders and trusses shall have full contact. If warped or deformed, the plates shall be heat straightened, planed, or corrected in some other way to produce accurate, even contact. If necessary for proper contact, bearing surfaces that will contact other metal surfaces shall be planed or milled. Surfaces of warped or deformed base and sole plates that will contact masonry shall be rough finished.

On the surface of expansion bearings, the cut of the planer shall be in the direction of expansion.

6-03.3(16) Abutting Joints

Abutting ends of compression members shall be faced accurately so that they bear evenly when in the structure. On built-up members, the ends shall be faced or milled after fabrication.

Ends of tension members at splices shall be rough finished to produce neat, close joints. A contact fit is not required.

6-03.3(17) End Connection Angles

On floorbeams and stringers, end connection angles shall be flush with each other and set accurately in relationship to the position and length of the member. Unless the Plans require it, end connection angles shall not be finished. If, however, faulty assembly requires them to be milled, milling shall not reduce thickness by more than $^{1}/_{16}$-inch.

6-03.3(18) Built Members

The various pieces forming one built member shall be straight and close fitting, true to detailed dimensions, and free from twists, bends, open joints, or other defects.

When fabricating curved girders, localized heat or the use of mechanical force shall not be used to bend the girder flanges about an axis parallel to girder webs.

6-03.3(19) Hand Holes

Hand holes, whether punched or cut with burning torches, shall be true to sizes and shapes shown in the Plans. Edges shall be true to line and ground smooth.

6-03.3(20) Lacing Bars

Unless the Plans state otherwise, ends of lacing bars shall be neatly rounded.

6-03.3(21) Plate Girders

6-03.3(21)A Web Plates

If web plates are spliced, clearance between plate ends shall not exceed $^{3}/_{8}$-inch.

6-03.3(21)B Vacant

6-03.3(21)C Web Splices and Fillers

Web splice plates and fillers under stiffeners shall fit within $^{1}/_{8}$-inch at each end. In lieu of the steel material specified in the Plans or Special Provisions, the Contractor may substitute ASTM A 1008 or ASTM A 1011 steel for all filler plates less than $^{1}/_{4}$-inch thickness, provided that the grade of filler plate steel meets or exceeds that of the splice plates.

6-03.3(22) Eyebars

Eyebars shall be straight, true to size, and free from twists or folds in the neck or head and from any other defect that would reduce their strength. Heads shall be formed by upsetting, rolling, or forging. Dies in use by the manufacturer may determine the shape of bar heads if the Engineer approves. Head and neck thickness shall not overrun by more than $^{1}/_{16}$-inch. Welds shall not be made in the body or head of any bar.

Each eyebar shall be properly annealed and carefully straightened before it is bored. Pinholes shall be located on the centerline of each bar and in the center of its head. Holes in bar ends shall be so precisely located that in a pile of bars for the same truss panel the pins may be inserted completely without driving. All eyebars made for the same locations in trusses shall be interchangeable.

6-03.3(23) Annealing

All eyebars shall be annealed by being heated uniformly to the proper temperature, then cooled slowly and evenly in the furnace. At all stages, the temperature of the bars shall be under full control.

Slight bends on secondary steel members may be made without heat. Crimped web stiffeners need no annealing.

6-03.3(24) Pins and Rollers

Pins and rollers shall be made of the class of forged steel the Plans specify. They shall be turned accurately to detailed dimensions, smooth, straight, and flawless. The final surface shall be produced by a finishing cut.

Pins and rollers 9-inches or less in diameter may either be forged and annealed or made of cold-finished carbon steel shafting.

Pins more than 9-inches in diameter shall have holes at least 2-inches in diameter bored longitudinally through their centers. Pins with inner defects will be rejected.

The Contractor shall provide pilot and driving nuts for each size of pin unless the Plans state otherwise.

6-03.3(24)A Boring Pin Holes

Pin holes shall be bored true to detailed dimensions, smooth and straight, and at right angles to the axis of the member. Holes shall be parallel with each other unless the Plans state otherwise. A finishing cut shall always be made.

The distance between holes shall not vary from detailed dimensions by more than $^{1}/_{32}$-inch. In tension members, this distance shall be measured from outside to outside of holes; in compression members, inside to inside.

6-03.3(24)B Pin Clearances

Each pin shall be $^{1}/_{50}$-inch smaller in diameter than its hole. All pins shall be numbered after being fitted into their holes in the assembled member.

6-03.3(25) Welding and Repair Welding

Welding and repair welding of all steel bridges shall comply with the AASHTO/AWS D1.5M/D1.5:2002 Bridge Welding Code. Welding and repair welding for all other steel fabrication shall comply with the AWS D1.1/D1.1M, latest edition, Structural Welding Code. The requirements described in the remainder of this section shall prevail whenever they differ from either of the above welding codes.

The Contractor shall weld structural steel only to the extent shown in the Plans. No welding, including tack and temporary welds shall be done in the shop or field unless the location of the welds is shown on the approved shop drawings or approved by the Engineer in writing.

Welding procedures shall be submitted for approval with shop drawings . The procedures shall specify the type of equipment to be used, electrode selection, preheat requirements, base materials, and joint details. When the procedures are not prequalified by AWS or AASHTO, evidence of qualification tests shall be submitted.

Welding shall not begin until after the Contractor has received the Engineer's approval of shop plans as required in Section 6-03.3(7). These plans shall include procedures for welding, assembly, and any heat-straightening or heat-curving.

Any welded shear connector longer than 8-inches may be made of two shorter shear connectors joined with full-penetration welds.

In shielded metal-arc welding, the Contractor shall use low-hydrogen electrodes.

In submerged-arc welding, flux shall be oven-dried at 550ºF for at least 2-hours, then stored in ovens held at 250ºF or more. If not used within 4-hours after removal from a drying or storage oven, flux shall be redried before use.

Preheat and interpass temperatures shall conform to the applicable welding code as specified in this section. When welding main members of steel bridges, the minimum preheat shall not be less than 100ºF.

If groove welds (web-to-web or flange-to-flange) have been rejected, they may be repaired no more than twice. If a third failure occurs, the Contractor shall:

1. Trim the members, if the Engineer approves, at least $^1/_2$-inch on each side of the weld; or
2. Replace the members at no expense to the Contracting Agency.

By using extension bars and runoff plates, the Contractor shall terminate groove welds in a way that ensures the soundness of each weld to its ends. The bars and plates shall be removed after the weld is finished and cooled. The weld ends shall then be ground smooth and flush with the edges of abutting parts.

The Contractor shall not:

1. Weld with electrogas or electroslag methods,
2. Weld nor flame cut when the ambient temperature is below 20ºF, or
3. Use coped holes in the web for welding butt splices in the flanges unless the Plans show them.

6-03.3(25)A Welding Inspection

The Contractor's inspection procedures, techniques, methods, acceptance criteria, and inspector qualifications for welding of steel bridges shall be in accordance with the AASHTO/AWS D1.5M/D1.5:2002 Bridge Welding Code. The Contractor's inspection procedures, techniques, methods, acceptance criteria, and inspector qualifications for welding of steel structures other than steel bridges shall be in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. The requirements described in the remainder of this section shall prevail whenever they differ from either of the above welding codes.

Nondestructive testing in addition to visual inspection shall be performed by the Contractor. Unless otherwise shown in the Plans or specified in the Special Provisions, the extent of inspection shall be as specified in this section. Testing and inspection shall apply to welding performed in the shop and in the field.

Visual Inspection

All welds shall be 100 percent visually inspected. Visual inspection shall be performed before, during, and after the completion of welding.

Radiographic Inspection

Complete penetration tension groove welds in highway bridges shall be 100 percent radiographically inspected. These welds include those in the tension area of webs, where inspection shall cover the greater of these two distances: (a) 15-inches from the tension flange, or (b) one third of the web depth. In addition, edge blocks conforming to the requirements of AASHTO/AWS D1.5M/D1.5:2002 Structural Welding Code Section 6.10.14 shall be used for radiographic inspection.

Ultrasonic Inspection

Complete penetration groove welds on plates thicker than $^{5}/_{16}$-inch in the following welded assemblies or structures shall be 100 percent ultrasonically inspected:

1. Welded connections and splices in highway bridges and earth retaining structures, excluding longitudinal butt joint welds in beam or girder webs.
2. Bridge bearings and modular expansion joints.
3. Sign bridges, cantilever sign structures, and bridge mounted sign brackets excluding longitudinal butt joint welds in beams.
4. Light, signal, and strain pole standards.

The testing procedure and acceptance criteria for tubular members shall conform to the requirements of the AWS D1.1/D1.1M latest edition, Structural Welding Code.

Magnetic Particle Inspection

1. Fillet and partial penetration groove welds:

   At least 30 percent of each size and type of fillet welds (excluding intermittent fillet welds) and partial penetration groove welds in the following welded assemblies or structures shall be tested by the magnetic particle method:

   a. Flange-to-web connections in highway bridges.
   b. End and intermediate pier diaphragms in highway bridges.
   c. Stiffeners and connection plates in highway bridges.
   d. Welded connections and splices in earth retaining structures.
   e. Boxed members of trusses.
   f. Bridge bearings and modular expansion joints.
   g. Sign bridges, cantilever sign structures, and bridge mounted sign brackets.
   h. Light, signal, and strain pole standards.
2. Longitudinal butt joint welds in beam and girder webs:

   At least 30 percent of each longitudinal butt joint weld in the beam and girder webs shall be tested by the magnetic particle method.
3. Complete penetration groove welds on plates $^{5}/_{16}$-inch or thinner shall be 100 percent tested by the magnetic particle method. Testing shall apply to both sides of the weld, if backing plate is not used.
4. The ends of each complete penetration groove weld at plate edges shall be tested by the magnetic particle method.

Where 100 percent testing is not required, the Engineer reserves the right to select the location(s) for testing.

If rejectable flaws are found in any test length of weld in Item 1 or 2 above, the full length of the weld or 5-feet on each side of the test length, whichever is less, shall be tested.

After the Contractor's welding inspection is complete, the Contractor shall allow the Engineer sufficient time to perform quality assurance ultrasonic welding inspection.

The Contractor shall maintain the radiographs and the radiographic inspection report in the shop until the last joint to be radiographed in that member is accepted by the radiographer representing the Contractor. Within two working days following this acceptance, the Contractor shall mail the film and two copies of the radiographic inspection report to the Materials Engineer, Department of Transportation, PO Box 47365, Olympia, WA 98504-7365.

6-03.3(26) Screw Threads

Screw threads shall be U.S. Standard and shall fit closely in the nuts.

6-03.3(27) High Strength Bolt Holes

At the Contractor's option under the conditions described in this section, holes may be punched or subpunched and reamed, drilled or subdrilled and reamed, or formed by numerically controlled drilling operations.

The hole for each high strength bolt shall be $^{1}/_{16}$-inch larger than the nominal diameter of the bolt.

In fabricating any connection, the Contractor may subdrill or subpunch the holes then ream full size after assembly or drill holes full size from the solid with all thicknesses of material shop assembled in the proper position. If the Contractor chooses not to use either of these methods, then the following shall apply:

1. Drill bolt holes in steel splice plates full size using steel templates.
2. Drill bolt holes in the main members of trusses, arches, continuous beam spans, bents, towers, plate girders, box girders, and rigid frames at all connections as follows:
    a. A minimum of 30 percent of the holes in one side of the connection shall be made full size using steel templates.
    b. A minimum of 30 percent of the holes in the second side shall be made full size assembled in the shop.
    c. All remaining holes may be made full size in unassembled members using steel templates.
3. Drill bolt holes in crossframes, gussets, lateral braces, and other secondary members full size using steel templates.

The Contractor shall submit for the Engineer's approval a detailed outline of the procedures proposed to accomplish the work from initial drilling through shop assembly.

6-03.3(27)A Punched Holes

For punched holes, die diameter shall not exceed punch diameter by more than $^{1}/_{16}$-inch. Any hole requiring enlargement to admit the bolt shall be reamed. All holes shall be cut clean with no torn or ragged edges. The Contracting Agency will reject components having poorly matched holes.

6-03.3(27)B Reamed and Drilled Holes

Reaming and drilling shall be done with short taper reamers or twist drills, producing cylindrical holes perpendicular to the member. Reamers and drills shall be directed mechanically, not hand-held. Connecting parts that require reamed or drilled holes shall be assembled and held securely as the holes are formed, then match-marked before disassembly. The Contractor shall provide the Engineer a diagram showing these match-marks. The Contracting Agency will reject components having poorly matched holes.

Burrs on outside surfaces shall be removed. If the Engineer requires, the Contractor shall disassemble parts to remove burrs.

If templates are used to ream or drill full-size connection holes, the templates shall be positioned and angled with extreme care and bolted firmly in place. Templates for reaming or drilling matching members or the opposite faces of one member shall be duplicates. All splice components shall be match-marked unless otherwise approved by the Engineer.

6-03.3(27)C Numerically Controlled Drilled Connections

In forming any hole described in Section 6-03.3(27), the fabricator may use numerically controlled (N/C) drilling or punching equipment if it meets the requirements in this subsection.

The Contractor shall submit for approval a detailed outline of proposed N/C procedures. This outline shall:

1. Cover all steps from initial drilling or punching through check assembly;
2. Include the specific members of the structure to be drilled or punched, hole sizes, locations of the common index and other reference points, makeup of check assemblies, and all other information needed to describe the process fully.

N/C holes may be drilled or punched to size through individual pieces, or may be drilled through any combination of tightly clamped pieces.

When the Engineer requires, the Contractor shall demonstrate that the N/C procedure consistently produces holes and connections meeting the requirements of these Specifications.

6-03.3(27)D Accuracy of Punched, Subpunched, and Subdrilled Holes

After shop assembly and before reaming, all punched, subpunched, and subdrilled holes shall meet the following standard of accuracy. At least 75 percent of the holes in each connection shall permit the passage of a cylindrical pin $^{1}/_{8}$-inch smaller in diameter than nominal hole size. This pin shall pass through at right angles to the face of the member without drifting. All holes shall permit passage of a pin $^{3}/_{16}$-inch smaller in diameter than nominal hole size. The Contracting Agency will reject any pieces that fail to meet these standards.

6-03.3(27)E Accuracy of Reamed and Drilled Holes

At least 85 percent of all holes in a connection of reamed or drilled holes shall show no offset greater than $^{1}/_{32}$-inch between adjacent thicknesses of metal. No hole shall have an offset greater than $^{1}/_{16}$-inch.

Centerlines from the connection shall be inscribed on the template and holes shall be located from these centerlines. Centerlines shall also be used for accurately locating the template relative to the milled or scribed ends of the members.

Templates shall have hardened steel bushing inserted into each hole. These bushings may be omitted, however, if the fabricator satisfies the Engineer (1) that the template will be used no more than 5 times, and (2) that use will produce no template wear.

Each template shall be at least $^{1}/_{2}$-inch thick. If necessary, thicker templates shall be used to prevent buckling and misalignment as holes are formed.

6-03.3(27)F Fitting for Bolting

Before drilling, reaming, and bolting begins, all parts of a member shall be assembled, well pinned, and drawn firmly together. If necessary, assembled pieces shall be taken apart to permit removal of any burrs or shavings produced as the holes are formed. The member shall be free from twists, bends, and other deformation.

In shop-bolted connections, contacting metal surfaces shall be sandblasted clean before assembly. Sandblasting shall meet the requirements of the SSPC Specifications for Commercial Blast Cleaning (SSPC-SP 6).

Any drifting done during assembly shall be no more than enough to bring the parts into place. Drifting shall not enlarge the holes or distort the metal.

6-03.3(28) Shop Assembly

6-03.3(28)A Method of Shop Assembly

Unless the contract states otherwise, the Contractor shall choose one of the five shop assembly methods described below that will best fit the proposed erection method. The Contractor shall obtain the Engineer's approval of both the shop assembly and the erection methods before work begins.

1. **Full Truss or Girder Assembly.** Each truss or girder is completely assembled over the full length of the superstructure.
2. **Progressive Truss or Girder Assembly.** Each truss or girder is assembled in stages longitudinally over the full length of the superstructure.
   a. For trusses: The first stage shall include at least three adjacent truss panels. Each truss panel shall include all of the truss members in the space bounded by the top and bottom chords and the horizontal distance between adjacent bottom chord Joints.
   b. For girders: The first stage shall include at least three adjacent girder shop sections. Shop sections are measured from the end of the girder to the first field splice or from field splice to field splice.
   c. For trusses and girders: After the first stage has been completed, each subsequent stage shall be assembled to include: two truss panels or girder shop sections of the previous stage and one or more truss panels or girder shop sections added at the advancing end. The previous stages shall be repositioned if necessary, and pinned to ensure accurate alignment. For straight sections of bridges without skews or tapers, girders in each subsequent stage may be assembled to include one girder shop section from the previous stage and one or more girder shop sections at the advancing end.

      If the bridge is longer than 150-feet, each longitudinal stage shall be at least 150-feet long, regardless of the length of individual continuous truss panels or girder shop sections.

      The Contractor may begin the assembly sequence at any point on the bridge and proceed in either or both directions from that point.

      Unless the Engineer approves otherwise, no assembly shall have less than three truss panels or girder shop sections.
3. **Full Chord Assembly.** The full length of each chord for each truss is assembled with geometric angles at the joints. Chord connection bolt holes are drilled/reamed while members are assembled. The truss web member connections are drilled/reamed to steel templates set by relating geometric angles to the chord lines.

   At least one end of each web member shall be milled or scribed at right angles to its long axis. The templates at both ends of the member shall be positioned accurately from the milled end or scribed line.
4. **Progressive Chord Assembly.** Adjacent chord sections are assembled in the same way as specified for Full Chord Assembly, using the procedure specified for Progressive Truss or Girder Assembly.
5. **Special Complete Structure Assembly.** All structural steel members (superstructure and substructure, including all secondary members) are assembled at one time.

6-03.3(28)B Check of Shop Assembly

The Contractor shall check each assembly for alignment, accuracy of holes, fit of milled joints, and other assembly techniques. Drilling or reaming shall not begin until the Engineer has given approval. If the Contractor uses N/C drilling, this approval must be obtained before the assembly or stage is dismantled.

6-03.3(29) Vacant

6-03.3(30) Painting

All painting shall be in accordance with Section 6-07.

6-03.3(30)A Vacant

6-03.3(30)B Vacant

6-03.3(30)C Erection Marks

Erection marks to permit identification of members in the field shall be painted on previously painted surfaces.

6-03.3(30)D Machine Finished Surfaces

As soon as possible and before they leave the shop, machine-finished surfaces on abutting chord splices, column splices, and column bases shall be covered with grease. After erection, the steel shall be cleaned and painted as specified.

All surfaces of iron and steel castings milled to smooth the surface shall be painted with the primer called for in the specified paint system.

While still in the shop, machine-finished surfaces and inaccessible surfaces of rocker or pin-type bearings shall receive the full paint system. Surfaces of pins and holes machine-finished to specific tolerances shall not be painted. But as soon as possible and before they leave the shop, they shall be coated with grease.

6-03.3(31) Alignment and Camber

Before beginning field bolting, the Contractor shall:

1. Adjust the structure to correct grade and alignment,
2. Regulate elevations of panel points (ends of floorbeams), and
3. Delay bolting at compression joints until adjusting the blocking to provide full and even bearing over the whole joint.

On truss spans, a slight excess camber will be permitted as the bottom chords are bolted. But camber and relative elevations of panel points shall be correct before the top chord joints, top lateral system, and sway braces are bolted.

6-03.3(31)A Measuring Camber

The Contractor shall provide the Engineer with a diagram for each truss that shows camber at each panel point. This diagram shall display actual measurements taken as the truss is being assembled.

6-03.3(32) Assembling and Bolting

To begin bolting any field connection or splice, the Contractor shall install and tighten to snug-tight enough bolts to bring all parts into full contact with each other prior to tightening these bolts to the specified minimum tension. “Snug-tight” means either the tightness reached by (l) a few blows from an impact wrench or (2) the full effort of a person using a spud wrench.

As erection proceeds, all field connections and splices for each member shall be securely drift pinned and bolted in accordance with 1 or 2 below before the weight of the member can be released or the next member is added. Field erection drawings shall specify pinning and bolting requirements that meet or exceed the following minimums:

1. **Joints in Normal Structures.** Fifty percent of the holes in a single field connection and fifty percent of the holes on each side of a single joint in a splice plate shall be filled with drift pins and bolts. Thirty percent of the filled holes shall be pinned. Seventy percent of the filled holes shall be bolted and tightened to snug-tight. Once all these bolts are snug-tight, each bolt shall be systematically tightened to the specified minimum tension. "Systematically tightened" means beginning with bolts in the most rigid part, which is usually the center of the joint, and working out to its free edges. The fully tensioned bolts shall be located near the middle of a single field connection or a single splice plate.
2. **Joints in Cantilevered Structures.** 75 percent of the holes in a single field connection and 75 percent of the holes on each side of a single joint in a splice plate shall be filled with drift pins and bolts. Fifty percent of the filled holes shall be pinned. Fifty percent of the filled holes shall be bolted and tightened to snug-tight. Once all these bolts are snug-tight, each bolt shall be systematically tightened to the specified minimum tension. The fully tensioned bolts shall be located near the middle of a single field connection or a single splice plate.

Drift pins shall be placed throughout each field connection and each field joint with the greatest concentration in the outer edges of a splice plate or member being bolted.

To complete a joint following the method listed above, the Contractor shall fill all remaining holes of the field connection or splice plate with bolts and tighten to snug-tight. Once all of these bolts are snug-tight, each bolt shall be systematically tightened to the specified minimum tension. After these bolts are tightened to the specified minimum tension, the Contractor shall replace the drift pins with bolts tightened to the specified minimum tension.

The Contractor may complete a field bolted connection or splice in a continuous operation before releasing the mass of the member or adding the next member. The Contractor shall utilize drift pins to align the connection. The alignment drift pins shall fill between 15 and 30 percent of the holes in a single field connection and between 15 and 30 percent of the holes on each side of a single joint in a splice plate. Once the alignment drift pins are in place, all remaining holes shall be filled with bolts and tightened to snug-tight starting from near the middle and proceeding toward the outer gage lines. Once all of these bolts are snug-tight, the Contractor shall systematically tighten all these bolts to the specified minimum tension. The Contractor shall then replace the drift pins with bolts. Each of these bolts shall be tightened to the specified minimum tension.

All bolts shall be placed with heads toward the outside and underside of the bridge. All high-strength bolts shall be installed and tightened before the falsework is removed.

The Contractor may erect metal railings as erection proceeds. But railings shall not be bolted or adjusted permanently until the falsework is released and the deck placed.

The Contractor shall not begin painting until the Engineer has inspected and accepted field bolting.

6-03.3(33) Bolted Connections

Bolts, nuts, hardened washers, and direct tension indicators shall meet the requirements of Section 9-06.5(3).

All bolted connections are friction type. Painted structures require Type 1 or Type 2 bolts. Unpainted structures require Type 3 bolts. AASHTO M 253 Type 1, 2, and 3 bolts shall not be galvanized or be used in contact with galvanized material.

Hardened washers are required under turned elements for connections using AASHTO M 164 and AASHTO M 253 bolts and, as required in the following:

1. Irrespective of the tightening method, hardened washers shall be used under both the head and the nut when AASHTO M 253 bolts are to be installed in structural carbon steel, as specified in Section 9-06.1.
2. Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened beveled washer shall be used to compensate for the lack of parallelism.

All galvanized nuts shall be lubricated with a lubricant containing a visible dye so a visual check for the lubricant can be made at the time of field installation. Black bolts shall be "oily" to the touch when installed. Weathered or rusted bolts and nuts shall be cleaned and relubricated prior to installation.

After assembly, bolted parts shall fit solidly together. They shall not be separated by washers, gaskets, or any other material. Assembled joint surfaces, including those next to bolt heads, nuts, and washers, shall be free of loose mill scale, burrs, dirt, and other foreign material that would prevent solid seating.

When all bolts in a joint are tight, each bolt shall carry at least the proof load shown in Table 3 below:

Table 3
Minimum Bolt Tension

Bolt Size (inches)	AASHTO M 164 (pounds)	AASHTO M 253 (pounds)
1/2	12,050	14,900
5/8	19,200	23,700
3/4	28,400	35,100
7/8	39,250	48,500
1	51,500	63,600
1 1/8	56,450	80,100
1 1/4	71,700	101,800
1 3/8	85,450	121,300
1 1/2	104,000	147,500

Tightening may be done by either the turn-of-nut or the direct-tension indicator method. Preferably, the nut shall be turned tight while the bolt is prevented from rotating. However, if required because of bolt entering and/or wrench operational clearances, tightening may be done by turning the bolt while the nut is prevented from rotating. Following are descriptions of the turn-of-nut and direct-tension-indicator methods:

1. **Turn-of-Nut Method.** Hardened steel washers shall be used under the turned elements. After a bolt in a connection or joint splice plate has been tightened to snug-tight and all specified bolting conditions satisfied, it shall be tightened to the specified minimum tension by rotating the amount specified in Table 4. Before final tightening, the Contractor shall match-mark with crayon or paint the outer face of each nut and the protruding part of the bolt. To ensure that this tightening method is followed, the Engineer will (1) observe as the Contractor installs and tightens all bolts and (2) inspect each match-mark.

Table 4
Turn-of-Nut Tightening Method Nut Rotational from Snug-Tight Condition

Bolt Length	Disposition of Outer Faces of Bolted Parts		
	Condition 1	***Condition 2***	***Condition 3***
L <= 4D	1/3 turn	1/2 turn	2/3 turn
4D < L<= 8D	1/2 turn	2/3 turn	5/6 turn
8D < L<= 12D	2/3 turn	5/6 turn	1 turn

Bolt length measured from underside of head to top of nut.

Condition 1 — both faces at right angles to bolt axis.

Condition 2 — one face at right angle to bolt axis, one face sloped no more than 1:20, without bevel washer.

Condition 3 — both faces sloped no more than 1:20 from right angle to bolt axis, without bevel washer.

Nut rotation is relative to the bolt regardless of which element (nut or bolt) is being turned. Tolerances permitted plus or minus 30 degrees (1/12 turn) for final turns of 1/2 turn or less; plus or minus 45 degrees (1/8 turn) for final turns of 2/3 turn or more.

D = nominal bolt diameter of bolt being tightened.

When bolt length exceeds 12D, the rotation shall be determined by actual tests in which a suitable tension device simulates actual conditions.

2. **Direct-Tension-Indicator Method.** DTIs shall not be used under the turned element. Direct-Tension-Indicators (DTIs) shall be placed under the bolt head with the protrusions facing the bolt head when the nut is turned. DTIs shall be placed under the nut with the protrusions facing the nut when the bolt is turned.

   DTIs shall be installed by two or more person crews with one individual preventing the element at the DTI from turning the measuring the gap of the DTI to determine the proper tension of the bolt.

   Three DTIs, per lot, shall be tested in a WSDOT approved bolt tension calibrator. The bolts shall be tensioned to 105 percent of the tension shown in Table 3. The test bolts shall not be tightened such that all of the DTI protrusions are completely crushed (all five openings with zero gap). The DTI gap between all protrusions shall be measured with a tapered feeler gage to the nearest 0.001-inch. All of the non-zero DTI gap measurements for the three test bolts shall be averaged. This average shall be used in the tightening of all the production bolts except as provided below.

All bolts in a connection shall be snug tightened prior to bringing any DTIs in the connection to full load. The maximum gap of the production bolt DTIs shall not be greater than the average test gap established above or 0.005-inch, whichever is less. The minimum gap of the production bolt DTIs may be zero (all five openings with zero gap).

The Contractor shall tension all bolts, inspecting all DTIs with a feeler gage, in the presence of the Engineer.

If a bolt, that has had its DTI brought to full load, loosens during the course of bolting the connection, the bolt shall have a new DTI installed and be retensioned. Reuse of the bolt and nut are subject to the provisions of this section.

AASHTO M 253 bolts and galvanized AASHTO M 164 bolts shall not be reused. Ungalvanized AASHTO M 164 bolts may be reused if approved by the Engineer. All bolts to be reused shall have their threads inspected for distortion by reinstalling the used nut on the bolt and turning the nut for the full length of the bolt threads by hand. Bolts to be reused shall be relubricated. Used bolts shall be subject to a rotational capacity test as specified in Section 6-03.3(33)A Pre-Erection Testing. Touching up or retightening previously tightened bolts which may have been loosened by the tightening of adjacent bolts shall not be considered as reuse, provided the snugging up continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 4.

6-03.3(33)A Pre-Erection Testing

High strength bolt assemblies (bolt, nut, and washer), black and galvanized, shall be subjected to a rotational capacity test (AASHTO M 164, Section 8.5) prior to any erection activity. Each combination of bolt production lot, nut lot, and washer lot shall be tested as an assembly. All tests shall be performed by the Contractor in the presence of the Engineer. Two specimens per lot shall be tested at the erection site immediately prior to installation, or whenever the Engineer deems it necessary. The bolt assemblies shall meet the following requirements.

1. Go through two times the required number of turns from snug tight condition as indicated in Table 4 of Section 6-03.3(33) without stripping, tensile, or shear failure. Rotation-capacity test shall be performed in a WSDOT approved bolt tension calibrator.
2. The maximum recorded tension shall be equal to or greater than 1.15 times the minimum bolt tension listed in Table 3 of Section 6.03.3(33).
3. The measured torque to produce the minimum bolt tension shall not exceed the value obtained by the following equation.

   Torque = 0.25 PD

Where:	Torque	=	Calculated Torque (foot-pounds)
	P	=	Measured Bolt Tension (pounds)
	D	=	Normal Bolt Diameter (feet)
4. Disassemble the torqued bolt and inspect for signs of failure. Failure is defined as any shear damage to the threads of the bolt or the nut or cracks in the body of the bolt. If either specimen fails, the lot of bolts will be rejected. Elongation of the bolt between the bolt head and the nut is not considered to be a failure.

6-03.3(33)B Bolting Inspection

The Contractor, in the presence of the Engineer, shall inspect the tightened bolt using an inspection torque wrench.

If the bolts to be installed are not long enough to fit in the Contracting Agency furnished tension calibrator, five bolts of the same grade, size, and condition as those under inspection shall be tested using Direct-Tension-Indicators (DTI) to measure bolt tension. This tension measurement test shall be done at least once each inspection day. The Contractor shall supply the necessary DTIs. The DTI shall be placed under the bolt head. A washer shall be placed under the nut, which shall be the element turned during the performance of this tension measurement test. Each bolt shall be tightened by any convenient means to the specified minimum tension as indicated by the DTI. The inspecting wrench shall then be applied to the tightened bolt to determine the torque required to turn the nut 5 degrees (approximately 1-inch at a 12-inch radius) in the tightening direction. The job inspection torque shall be taken as the average of three values thus determined after rejecting the high and low values.

Five bolts (provided by the Contractor) of the same grade, size, and condition as those under inspection shall be placed individually in a Contracting Agency furnished tension calibrator to measure bolt tension. This calibration operation shall be done at least once each inspection day. There shall be a washer under the part turned in tightening each bolt if washers are used on the structure. In the calibrated device, each bolt shall be tightened by any convenient means to the specified tension. The inspecting wrench shall then be applied to the tightened bolt to determine the torque required to turn the nut or head 5 degrees (approximately 1-inch at a 12-inch radius) in the tightening direction. The job-inspection torque shall be taken as the average of three values thus determined after rejecting the high and low values.

Ten percent (at least two) of the tightened bolts on the structure represented by the test bolts shall be selected at random in each connection. The job-inspection torque shall then be applied to each with the inspecting wrench turned in the tightening direction. If this torque turns no bolt head or nut, the Contracting Agency will accept the connection as being properly tightened. But if the torque turns one or more bolt heads or nuts, the job-inspection torque shall then be applied to all bolts in the connection. Any bolt whose head or nut turns at this stage shall be tightened and reinspected. The Contractor may, however, retighten all the bolts in the connection and resubmit it for inspection.

6-03.3(34) Adjusting Pin Nuts

All pin nuts shall be tightened thoroughly. The pins shall be placed so that members bear fully and evenly on the nuts. The pins shall have enough thread to allow burring after the nuts are tightened.

6-03.3(35) Setting Anchor Bolts

Anchor bolts shall be set in masonry as required in Section 6-02.3(18). Anchor bolts shall be grouted in after the shoes, masonry plates, and keeper plates have been set and the span or series of continuous spans are completely erected and adjusted to line and camber.

6-03.3(36) Setting and Grouting Masonry Plates

The following procedure applies to masonry plates for all steel spans, including shoes, keeper plates, and turning racks on movable bridges.

To set masonry plates, the Contractor shall:

1. Set masonry plates on the anchor bolts;
2. Place steel shims under the masonry plates to position pin centers or bearings to line and grade and in relationship to each other. Steel shims shall be no more than $2^1/_2$-inches square and placed under plate webs;
3. Level the bases of all masonry plates;
4. Draw anchor bolt nuts down tight;
5. Recheck pin centers or bearings for alignment; and
6. Leave at least $^3/_4$-inch of space under each masonry plate for grout.

After the masonry plates have been set and the span or series of continuous spans are completely erected and swung free, the space between the top of the masonry and the top of the concrete bearing seat shall be filled with grout. Main masonry plates for cantilever spans shall be set and grouted in before any steel work is erected.

Grout mixture and placement shall be as required in Section 6-02.3(20).

6-03.3(37) Setting Steel Bridge Bearings

Masonry plates, shoes, and keeper plates of expansion bearings shall be set and adjusted to center at a normal temperature of 64ºF. Adjustment for an inaccuracy in fabricated length shall be made after dead-load camber is out.

6-03.3(38) Placing Superstructure

The Contractor shall place no superstructure load on finished piers or abutments until the Engineer allows. Normally, this concrete-hardening interval requires at least 12 days.

6-03.3(39) Swinging the Span

No forms, steel reinforcing bars, or concrete roadway slabs shall be placed on steel spans until the spans swing free on their supports and elevations recorded. No simple span or any series of continuous spans will be considered as swinging free until all temporary supports have been released. Forms, reinforcing steel, or concrete roadway slabs shall not be placed on any simple or continuous span steel girder bridge until all its spans are adjusted and its masonry plates, shoes, and keeper plates grouted. For this specification, the structure shall be considered as continuous across hinged joints.

After the falsework is released (spans swung free) the masonry plates, shoes, and keeper plates are grouted, and before any load is applied, the Engineer will (or, if the Contractor is specified as responsible for surveying, the Contractor shall) measure elevations at the tenth points along the tops of girders and floorbeams.

The Engineer will compare steel mass camber elevations with the elevations measured above, and will furnish the Contractor with new dead-load camber dimensions.

6-03.3(40) Draining Pockets

The Contractor shall provide enough holes to drain all water from pockets in trusses, girders, and other members. Unless shown on approved shop plans, drain holes shall not be drilled without the written approval of the Engineer.

All costs related to providing drain holes shall be included in the unit contract prices for structural or cast steel.

6-03.3(41) Floorbeam Protection

Each floorbeam that supports a concrete slab joint shall be coated on its top and flange edges with a heavy mop of roofing grade asphalt, applied hot. This asphalt shall conform to ASTM D 312 (not mineral stabilized). A protective covering of asphalt coated glass fiber sheet (ASTM D 4601 Type 1 non-perforated) shall be placed over the hot coat of asphalt. This combination coating shall be applied over the shop paint. It shall take the place of the two field coats of paint specified for other parts of the structural steel.

6-03.3(42) Surface Condition

As the structure is erected, the Contractor shall keep all steel surfaces clean and free from dirt, concrete, mortar, oil, paint, grease, and other stain-producing foreign matter. Any surfaces that become stained shall be cleaned as follows:

> Painted steel surfaces shall be cleaned by methods required for the type of staining. The method shall be submitted to the Engineer for approval.
>
> Unpainted steel surfaces shall be cleaned by sandblasting. Sandblasting to remove stains on publicly visible surfaces shall be done to the extent that, in the Engineers opinion, the uniform weathering characteristics of the structure are preserved.

6-03.3(43) Castings, Steel Forgings, and Miscellaneous Metals

Castings, steel forgings, and miscellaneous metals shall be built to comply with Section 9-06.

6-03.3(43)A Shop Construction, Castings, Steel Forgings, and Miscellaneous Metals

This section's requirements for structural steel (including painting requirements) shall also apply to castings, steel forgings, and miscellaneous metals.

Castings shall be:

1. True to pattern in form and dimensions;
2. Free from pouring faults, sponginess, cracks, blow holes, and other defects in places that would affect strength, appearance, or value;
3. Clean and uniform in appearance;
4. Filleted boldly at angles; and
5. Formed with sharp and perfect arises.

Iron and steel castings and forgings shall be annealed before any machining, unless the Plans state otherwise.

6-03.4 Measurement

Structural carbon steel, structural low alloy steel, and structural high strength steel will not be measured but will be paid for on a lump sum basis as described in Section 6-03.5.

Cast or forged metal (kind) or copper seals shown in the Plans will be measured by the pound or will be paid for on a lump sum basis, whichever is shown on the proposal.

6-03.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

"Structural Carbon Steel", lump sum.

The lump sum contract price for "Structural Carbon Steel" shall be full pay for all costs in connection with furnishing all materials, labor, tools, and equipment necessary for the manufacture, fabrication, transportation, erection, and painting of all structural carbon steel used in the completed structure, including the providing of such other protective coatings or treatment as may be shown in the Plans or specified in the Special Provisions.

For steel structures, the estimated weight of the structural carbon steel in the project will be shown in the Plans or in the Special Provisions. In the event any change in the Plans is made which will affect the weight of materials to be furnished, payment for the additional structural carbon steel required as a result of the change in the Plans will be made at a unit price per pound obtained by dividing the Contractor's lump sum bid for structural carbon steel by the total estimated weight of structural carbon steel shown in the Plans or in the Special Provisions.

Reductions in weight due to a change in the Plans will be made at the same rate as determined above and will be deducted from payments due the Contractor.

Prospective bidders shall verify the estimated weight of structural carbon steel before submitting a bid. No adjustment other than for approved changes will be made in the lump sum bid even though the actual weight may deviate from the stated estimated weight.

For concrete and timber structures, where the structural carbon steel is a minor item, no estimated weight will be given for the structural carbon steel. In the event any change in the Plans is necessary which will affect the weight of material to be furnished for this type of structure, the payment or reduction for the revision in quantity will be made at a unit price per pound obtained by dividing the Contractor's lump sum bid for the structural carbon steel by the calculated weight of the original material. The calculated weight will be established by the Engineer and will be based on an estimated weight of 490 pounds per cubic foot for steel.

Any change in the Plans which affects the weight of material to be furnished as provided herein will be subject to the provisions of Section 1-04.4.

"Structural Low Alloy Steel", lump sum.

"Structural High Strength Steel", lump sum.

Payment for "Structural Low Alloy Steel" and "Structural High Strength Steel" will be made on the same lump sum basis as specified for structural carbon steel.

"(Cast or Forged) Steel", lump sum or per pound.

"(Cast, Malleable, or Ductile) Iron", lump sum or per pound.

"Cast Bronze", lump sum or per pound.

Payment for "(Cast or Forged) Steel", "(Cast, Malleable or Ductile) Iron", and "Cast Bronze" will be made at the lump sum or per pound contract prices as included in the proposal.

For the purpose of payment, such minor items as bearing plates, pedestals, forged steel pins, anchor bolts, field bolts, shear connectors, etc., unless otherwise provided, shall be considered as structural carbon steel even though made of other materials.

When no bid item is included in the proposal and payment is not otherwise provided, the castings, forgings, miscellaneous metal, and painting shall be considered as incidental to the construction, and all costs therefore shall be included in the unit contract prices for the payment items involved and shown.

6-04 TIMBER STRUCTURES

6-04.1 Description

This work is the building of any structure or parts of structures (except piling) made of treated timber, untreated timber, or both. The Contractor shall erect timber structures on prepared foundations. The structures shall conform to the dimensions, lines, and grades required by the Plans, the Engineer, and these Specifications.

Any part of a timber structure made of nontimber materials shall comply with the sections of these Specifications that govern those materials.

6-04.2 Materials

Materials shall meet the requirements of the following sections:

Structural Steel and Related Material	9-06
Bolts, Washers, Other Hardware	9-06.22
Paints	9-08
Timber and Lumber	9-09

6-04.3 Construction Requirements

6-04.3(1) Storing and Handling Material

At the work site, the Contractor shall store all timber and lumber in piles. Weeds and rubbish under and around these piles shall have been removed before the lumber is stacked.

Untreated lumber shall be open stacked at least 12-inches above the ground. It shall be piled to shed water and prevent warping.

Treated timber shall be:

1. Cut, framed, and bored (whenever possible) before treatment;
2. Close stacked and piled to prevent warping;
3. Covered against the weather if the Engineer requires it;
4. Handled carefully to avoid sudden drops, broken outer fibers, and surface penetration or bruising with tools; and
5. Lifted and moved with rope or chain slings (without use of cant dogs, peaveys, hooks, or pike poles).

6-04.3(2) Workmanship

The Contractor shall employ only competent bridge carpenters. All their work shall be true and exact. Nails and spikes shall be driven with just enough force to leave heads flush with wood surfaces. The Contractor shall discharge any worker who displays poor workmanship by leaving deep hammer marks in wood surfaces. Workmanship on metal parts shall comply with requirements for steel structures.

6-04.3(3) Shop Details

The Contractor shall provide the Engineer with six sets of shop detail plans for all treated timber. These plans shall show dimensions for all cut, framed, or bored timbers. The Engineer will return to the Contractor one set of approved or corrected plans. No material shall be framed or bored until the Engineer approves the plans. Plans shall be drawn on sheets that conform to the sizes required in Section 1-05.3.

6-04.3(4) Field Treatment of Cut Surfaces, Bolt Holes, and Contact Surfaces

All cut surfaces, bolt holes, and contact surfaces shall be treated in accordance with Section 9-09.3 for all timber and lumber requiring preservative treatment.

All cuts and abrasions in treated piles or timbers shall be trimmed carefully and treated in accordance with Section 9-09.3.

6-04.3(5) Holes for Bolts, Dowels, Rods, and Lag Screws

Holes shall be bored:

1. For drift pins and dowels — with a bit $^{1}/_{16}$-inch smaller in diameter than the pins and dowels.
2. For truss rods or bolts — with a bit the same diameter as the rods or bolts.
3. For lag screws — in two parts: (a) with the shank lead hole the same diameter as the shank and as deep as the unthreaded shank is long; and (b) with the lead hole for the threaded part approximately two thirds of the shank diameter.

6-04.3(6) Bolts, Washers, and Other Hardware

Bolts, dowels, washers, and other hardware, including nails, shall be black or galvanized as specified in the Plans, but if not so specified shall be galvanized when used in treated timber structures.

Washers of the size and type specified shall be used under all bolt heads and nuts that would otherwise contact wood.

All bolts shall be checked by burring the threads after the nuts have been finally tightened. Vertical bolts shall have nuts on the lower ends.

Wherever bolts fasten timber to timber, to concrete, or to steel, the members shall be bolted tightly together at installation and retightened just before the Contracting Agency accepts the work. These bolts shall have surplus threading of at least $^{3}/_{8}$-inch per foot of timber thickness to permit future tightening.

6-04.3(7) Countersinking

Countersinking shall be done wherever smooth faces are required. Each recess shall be treated in accordance with Section 9-09.3.

6-04.3(8) Framing

The Contractor shall cut and frame lumber and timber to produce close-fitting, full-contact joints. Each mortise shall be true to size for its full depth, and its tenon shall fit it snugly. Neither shimmed nor open joints are permitted.

6-04.3(9) Framed Bents

Mudsills shall be of pressure-treated timber, firmly and evenly bedded to solid bearing, and tamped in place.

Concrete pedestals that support framed bents shall be finished so that sills will bear evenly on them. To anchor the sills, the Contractor shall set dowels in the pedestals when they are cast. The dowels shall be at least $^{3}/_{4}$-inch in diameter and protrude at least 6-inches above the pedestal tops. Pedestal concrete shall comply with Section 6-02.

Each sill shall rest squarely on mudsills, piles, or pedestals. It shall be drift-bolted to mudsills or piles with $^{3}/_{4}$-inch diameter or larger bolts that extend at least 6-inches into them. When possible, the Contractor shall remove any earth touching the sills to permit free air circulation around them.

Each post shall be fastened to sills with $^3/_4$-inch diameter or larger dowels that extend at least 6-inches into the post.

6-04.3(10) Caps

Timber caps shall rest uniformly across the tops of posts or piles and cap ends shall be aligned evenly. Each cap shall be fastened with a drift bolt $^3/_4$-inch in diameter or larger that penetrates the post or pile at least 9-inches. The bolt shall be approximately in the center of the pile or post.

If the roadway grade exceeds 2 percent, each cap shall be beveled to match the grade.

6-04.3(11) Bracing

When pile bents are taller than 10-feet, each shall be braced transversely and every other pair shall be braced longitudinally. No single cross-bracing shall brace more than 20-feet of vertical distance on the piles. If the vertical distance exceeds 20-feet, more than one cross-bracing shall be used. Each brace end shall be bolted through the pile, post, or cap with a bolt $^3/_4$-inch in diameter or larger. Other brace/pile intersections shall be bolted or boat-spiked as the Plans require. Cross-bracing shall lap both upper or lower caps and shall be bolted to the caps or sills at each end.

6-04.3(12) Stringers

All stringers that carry laminated decking or vary more than $^1/_8$-inch in depth shall be sized to an even depth at bearing points. Outside stringers shall be butt jointed and spliced. Interior stringers shall be lapped so that each rests over the full width of the cap or floorbeam at each end. Except on sharp horizontal and vertical curves, stringers may cover two spans. In this case, joints shall be staggered and the stringers either toenailed or drift bolted as the Plans require. To permit air circulation on untreated timber structures, the ends of lapped stringers shall be separated. This separation shall be done by fastening across the lapping face a 1-inch by 3-inch wood strip cut 2-inches shorter than the depth of the stringer.

Any cross-bridging or solid bridging shall be neatly and accurately framed, then securely toenailed at each end (with two nails for cross-bridging and four nails for solid bridging). The Plans show bridging size and spacing.

6-04.3(13) Wheel Guards and Railings

Wheel guards and railings shall be built as Section 6-06.3(1) requires.

6-04.3(14) Single-Plank Floors

Single-plank floors shall be made of a single thickness of plank on stringers or joists. Unless the Engineer directs otherwise, the planks shall be:

1. Laid heart side down with tight joints,
2. Spiked to each joist or nailing strip with at least two spikes that are at least 4-inches longer than the plank thickness,
3. Spiked at least $2^1/_2$-inches from the edges,
4. Cut off on a straight line parallel to the centerline of the roadway,
5. Arranged so that no adjacent planks vary in thickness by no more than $^1/_{16}$-inch, and
6. Surfaced on one side and one edge (S1S1E) unless otherwise specified.

6-04.3(15) Laminated Floors

The strips shall be placed on edge and shall be drawn down tightly against the stringer or nailing strip and the adjacent strip and, while held in place, shall be spiked. Each strip shall extend the full width of the deck, unless some other arrangement is shown in the Plans or permitted by the Engineer.

Each strip shall be spiked to the adjacent strip at intervals of not more than 2-feet, the spikes being staggered 8-inches in adjacent strips. The spikes shall be of sufficient length to pass through two strips and at least halfway through the third. In addition, unless bolting is specified in the Plans, each strip shall be toenailed to alternate stringers with 40d common nails and adjacent strips shall be nailed to every alternate stringer. The ends of all pieces shall be toenailed to the outside stringer. The ends of the strips shall be cut off on a true line parallel to the centerline of the roadway. When bolts are used to fasten laminated floors to stringers, the bolts shall be placed at the spacing shown in the Plans, and the pieces shall be drawn down tightly to the bolting strips. The bolt heads shall be driven flush with the surface of the deck. Double nuts or single nuts and lock nuts shall be used on all bolts. The strips shall be spiked together in the same manner as specified above.

6-04.3(16) Plank Subfloors for Concrete Decks

Any plank subfloor shall be laid surfaced side down with close joints at right angles to the centerline of the roadway. Planks shall be spiked in place as required in Section 6-04.3(14).

Floor planks shall be treated in accordance with Section 9-09.3.

6-04.3(17) Trusses

Completed trusses shall show no irregularities of line. From end to end, chords shall be straight and true in horizontal projection. In vertical projection they shall show a smooth curve through panel points that conforms to the correct camber. The Engineer will reject any pieces cut unevenly or roughly at bearing points. Before placement of the hand railing, the Contractor shall complete all trusses, swing them free of their falsework, and adjust them for line and camber (unless the Engineer directs otherwise).

6-04.3(18) Painting

Section 6-07.3(3) governs painting of timber structures.

6-04.4 Measurement

The criteria in Section 6-03.4 will be used to determine the weight of structural metal other than hardware.

Timber and lumber (treated or untreated) will be measured by the 1,000 board feet (MBM), using nominal thicknesses and widths. Lengths will be actual lengths of individual pieces in the finished structure with no deduction for daps, cuts, or splices. To measure laminated timber decking, the Contracting Agency will use the number and after-dressing sizes of pieces required in the Plans. The length of each lamination shall be the length remaining in the finished structure.

6-04.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

1. "Timber and Lumber (untreated or name treatment)", per MBM.
2. "Structural Metal", lump sum.

Where no item for structural metal is included in the proposal, full pay for furnishing and placing metal parts shall be included in the unit contract price per MBM for "Timber and Lumber".

When no bid item is included in the proposal and is not otherwise provided, painting shall be considered as incidental to the construction, and all costs therefore shall be included in the unit contract prices for the payment items involved and shown.

6-05 PILING

6-05.1 Description

This work consists of furnishing and driving piles (timber, precast concrete, cast-in-place concrete, and steel) of the sizes and types the Contract or the Engineer require. This work also includes cutting off or building up piles when required. In furnishing and driving piles, the Contractor shall comply with the requirements of this section, the Contract, and the Engineer.

6-05.2 Materials

Materials shall meet the requirements of the following sections:

Reinforcing Steel	9-07
Prestressing Steel	9-07.10
Timber Piling	9-10.1
Concrete Piling	9-10.2
Cast-in-Place Concrete Piling	9-10.3
Steel Pile Tips and Shoes	9-10.4
Steel Piling	9-10.5

6-05.3 Construction Requirements

6-05.3(1) Piling Terms

Concrete Piles. Concrete piling may be precast or precast-prestressed concrete, or steel casings driven to the ultimate bearing capacity called for in the Contract which are filled with concrete (cast-in-place) after driving.

Steel Piles. Steel piles may be open-ended or closed-ended pipe piles, or H-piles.

Overdriving. Over-driving of piles occurs when the ultimate bearing capacity calculated from the equation in Section 6-05.3(12), or the wave equation if applicable, exceeds the ultimate bearing capacity required in the Contract in order to reach the minimum tip elevation specified in the Contract, or as required by the Engineer.

Maximum Driving Resistance. The maximum driving resistance is either the pile ultimate bearing capacity, or ultimate bearing capacity plus overdriving to reach minimum tip elevation as specified in the Contract, whichever is greater.

Wave Equation Analysis. Wave equation analysis is an analysis performed using the wave equation analysis program (WEAP) with a version dated 1987 or later. The wave equation may be used as specified herein to verify the Contractor's proposed pile driving system. The pile driving system includes, but is not necessarily limited to, the pile, the hammer, the helmet, and any cushion. The wave equation may also be used by the Engineer to determine pile driving criteria as may be required in the Contract.

Ultimate Bearing Capacity. Ultimate bearing capacity refers to the vertical load carrying capacity (in units of force) of a pile as determined by the equation in Section 6-05.3(12), the wave equation analysis, pile driving analyzer and CAPWAP, static load test, or any other means as may be required by the Contract, or the Engineer.

Allowable Bearing Capacity. Allowable bearing capacity is the ultimate bearing capacity divided by a factor of safety. The Contract may state the factor of safety to be used in calculating the allowable bearing capacity from the ultimate bearing capacity. In the absence of a specified factor of safety, a value of three (3) shall be used.

Rated Hammer Energy. The rated energy represents the theoretical maximum amount of gross energy that a pile driving hammer can generate. The rated energy of a pile driving hammer will be stated in the hammer manufacturer's catalog or specifications for that pile driving hammer.

Developed Hammer Energy. The developed hammer energy is the actual amount of gross energy produced by the hammer for a given blow. This value will never exceed the rated hammer energy. The developed energy may be calculated as the ram weight times the drop (or stroke) for drop, single acting hydraulic, single acting air/steam, and open-ended diesel hammers. For double acting hydraulic and air/steam hammers, the developed hammer energy shall be calculated from ram impact velocity measurements or other means approved by the Engineer. For closed-ended diesel hammers, the developed energy shall be calculated from the measured bounce chamber pressure for a given blow. Hammer manufacturer calibration data may be used to correlate bounce chamber pressure to developed hammer energy. For a single acting diesel hammer the developed energy is determined using the blows per minute.

Transferred Hammer Energy. The transferred hammer energy is the amount of energy transferred to the pile for a given blow. This value will never exceed the developed hammer energy. Factors that cause transferred hammer energy to be lower than the developed hammer energy include friction during the ram down stroke, energy retained in the ram and helmet during rebound, and other impact losses. The transferred energy can only be measured directly by use of sensors attached to the pile. A pile driving analyzer (PDA) may be used to measure transferred energy.

Pile Driving Analyzer. A pile driving analyzer (PDA) is a device which can measure the transferred energy of a pile driving system, the compressive and tensile stresses induced in the pile due to driving, the bending stresses induced by hammer misalignment with the pile, and estimate the ultimate capacity of a pile at a given blow.

Pile Driving System. The pile driving system includes, but is not necessarily limited to, the hammer, leads, helmet or cap, cushion and pile.

Helmet. The helmet, also termed the cap, drive cap, or driving head, is used to transmit impact forces from the hammer ram to the pile top as uniformly as possible across the pile top such that the impact force of the ram is transmitted axially to the pile. The term helmet can refer to the complete impact force transfer system, which includes the anvil or striker plate, hammer cushion and cushion block, and a pile cushion if used, or just the single piece unit into which these other components (anvil, hammer cushion, etc.) fit. The helmet does not include a follower, if one is used. For hydraulic hammers, the helmet is sometimes referred to as the anvil.

Hammer Cushion. The hammer cushion is a disk of material placed on top of the helmet but below the anvil or striker plate to relieve impact shock, thus protecting the hammer and the pile.

Pile Cushion. The pile cushion is a disk of material placed between the helmet and the pile top to relieve impact shock, primarily to protect the pile.

Follower. A follower is a structural member placed between the hammer assembly, which includes the helmet, and the pile top when the pile head is below the reach of the hammer.

Pile Driving Refusal. Pile driving refusal is defined as 15 blows per inch for the last 4-inches of driving. This is the maximum blow count allowed during overdriving.

Minimum Tip Elevation. The minimum tip elevation is the elevation to which the pile tip must be driven. Driving deeper in order to obtain the required ultimate bearing capacity may be required.

6-05.3(2) Ordering Piling

The Contractor shall order all piling (except cast-in-place concrete and steel piles) from an itemized list the Engineer will provide. This list, showing the number and lengths of piles required, will be based on test-pile driving (or other) data. The list will show lengths below the cutoff point. The Contractor shall supply (and bear the cost of supplying) any additional length required for handling or driving.

The Contractor shall assume all responsibility for buying more or longer piles than those shown on the list provided by the Engineer. All piles purchased on the basis of the Engineer's list but not used in the finished structure shall become the property of the Contracting Agency. The Contractor shall deliver these as the Engineer directs. The Contractor shall keep pile cutoffs that are 8-feet or under and any longer ones the Contracting Agency does not require.

When ordering steel casings for cast-in-place concrete and steel piling, the Contractor shall base lengths on information derived from driving test piles and from subsurface data. The Contractor shall also select the wall thickness of steel piles or steel casings for cast-in-place piles which will be necessary to prevent damage during driving and handling. The selection of wall thickness for steel piles or steel casings shall also consider the effects of lateral pressures from the soil or due to driving of adjacent piles. Steel piles and steel casings must be strong and rigid enough to resist these pressures without deforming or distorting. The Contractor shall select the wall thickness based on information derived from test piles, subsurface data and/or wave equation analysis. Wave equation analysis is required prior to ordering piling for piles with specified ultimate bearing capacities of 300 tons or greater. If a wave equation analysis is performed, the Contractor shall base the selection of wall thickness on the maximum driving resistance identified in the Contract to reach the minimum tip elevation, if the maximum driving resistance is greater than the specified ultimate bearing capacity and if a minimum tip elevation is specified. The wave equation analysis shall be submitted by the Contractor as required in Section 6-05.3(9)A. The Engineer will not supply any list for piling of these types.

The Contractor shall obtain the Engineer's approval of pile dimensions before any steel casings or steel piles are ordered or shipped.

6-05.3(3) Manufacture of Precast Concrete Piling

Precast concrete piles shall consist of concrete sections reinforced to withstand handling and driving stresses. These may be reinforced with deformed steel bars or prestressed with steel strands. The Plans show dimensions and details. If the Plans require piles with square cross-sections, the corners shall be chamfered 1-inch.

Precast or prestressed piles shall meet the requirements of the Standard Plans.

Temporary stress in the prestressing reinforcement of prestressed piles (before loss from creep and shrinkage) shall be 75 percent of the minimum ultimate tensile strength. (For short periods during manufacture, the reinforcement may be overstressed to 80 percent of ultimate tensile strength if stress after transfer to concrete does not exceed 75 percent of that strength.)

Prestressed concrete piles shall have a final (effective) prestress of at least 1,000 psi.

Unless the Engineer approves splices, all piles shall be full length.

The Contracting Agency intends to perform Quality Assurance Inspection. By its inspection, the Contracting Agency intends only to facilitate the work and verify the quality of that work. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and workmanship.

6-05.3(3)A Casting and Stressing

Reinforcing bars, hoops, shoes, etc. shall be placed as shown in the Contract, with all parts securely tied together and placed to the specified spacing. No concrete shall be poured until all reinforcement is in place in the forms.

The Contractor shall perform quality control inspection. The manufacturing plant for precast concrete piling shall be certified by the Precast/Prestressed Concrete Institute's Plan Certification Program for the type of precast piling to be produced and shall be approved by WSDOT as a Certified Precast Concrete Fabricator prior to start of production. WSDOT Certification will be established or renewed during the annual precast plant review and approval process.

Prior to the start of production of the piling, the Contractor shall advise the Engineer of the production schedule. The Contractor shall give the Inspector safe and free access to the work. If the Inspector observes any nonspecification work or unacceptable quality control practices, the Inspector will advise the plant manager. If the corrective action is not acceptable to the Engineer, the piling(s) will be rejected.

In casting concrete piles, the Contractor shall:

1. Cast them either vertically or horizontally;
2. Use metal forms (unless the Engineer approves otherwise) with smooth joints and inside surfaces that can be reached for cleaning after each use;
3. Brace and stiffen the forms to prevent distortion;
4. Place concrete continuously in each pile, guarding against horizontal or diagonal cleavage planes;
5. Ensure that the reinforcement is properly embedded;
6. Use internal vibration around the reinforcement during concrete placement to prevent rock pockets from forming; and
7. Cast test cylinders with each set of piles as concrete is placed.

Forms shall be metal and shall be braced and stiffened to retain their shape under pressure of wet concrete. Forms shall have smooth joints and inside surfaces easy to reach and clean after each use. That part of a form which will shape the end surface of the pile shall be a true plane at right angles to the pile axis.

Each pile shall contain a cage of nonprestressed reinforcing steel. The Contractor shall follow the Contract in the size and location of this cage, and shall secure it in position during concrete placement. Spiral steel reinforcing shall be covered by at least $1^1/_2$-inches of concrete measured from the outside pile surface.

Prestressing steel shall be tensioned as required in Section 6-02.3(25)C.

The Plans specify tensioning stress for strands or wires. Tension shall be measured by jack pressure as described in Section 6-02.3(25)C. Mechanical locks or anchors shall temporarily maintain cable tension. All jacks shall have hydraulic pressure gauges (accurately calibrated and accompanied by a certified calibration curve no more than 180 days old) that will permit stress calculations at all times.

All tensioned piles shall be pretensioned. Post-tensioning is not allowed.

The Contractor shall not stress any pile until test cylinders made with it reach a compressive strength of at least 3,300 psi.

6-05.3(3)B Finishing

As soon as the forms for precast concrete piles are removed, the Contractor shall fill all holes and irregularities with 1:2 mortar. That part of any pile that will be underground or below the low-water line and all parts of any pile to be used in salt water or alkaline soil shall receive only this mortar treatment. That part of any pile that will show above the ground or water line shall be given a Class 2 finish as described in Section 6-02.3(14)B.

6-05.3(3)C Curing

Precast Concrete Piles. The Contractor:

1. Shall keep the concrete continuously wet with water after placement for at least ten days with Type I or II Portland cement or at least three days with Type III.
2. Shall remove side forms no sooner than 24 hours after concrete placement, and then only if the surrounding air remains at no less than 50°F for five days with Type I or II Portland cement or three days with Type III.
3. May cure precast piles with saturated steam or hot air, as described in Section 6-02.3(25)D, provided the piles are kept continuously wet until the concrete has reached a compressive strength of 3,300 psi.

Precast-Prestressed Concrete Piles. These piles shall be cured as required in Section 6-02.3(25)D.

6-05.3(4) Manufacture of Steel Casings for Cast-in-Place Concrete Piles

The diameter of steel casings shall be as specified in the Contract. Spiral welded steel pile casings are not allowed for steel pile casings greater that 24-inches in diameter. A full penetration groove weld with a maximum $^1/_{16}$-inch offset between welded edges is required.

6-05.3(5) Manufacture of Steel Piles

Steel piles shall be made of rolled steel H-pile sections, steel pipe piles, or of other structural steel sections described in the Contract. Spiral welded steel pile casings are not allowed for steel pipe piles greater that 24-inches in diameter. A full penetration groove weld with a maximum $^1/_{16}$-inch offset between welded edges is required.

6-05.3(6) Splicing Steel Casings and Steel Piles

The Engineer will normally permit steel piles and steel casings for cast-in-place concrete piles to be spliced. But in each case, the Contractor must obtain approval on the need and the method for splicing. Welded splices shall be spaced at a minimum distance of 10-feet. Only welded splices will be permitted.

Splice welds shall comply with Section 6-03.3(25) and AWS D1.1 Structural Welding Code. Splicing of steel piles shall be performed in accordance with an approved weld procedure. The Contractor shall submit a weld procedure to the Engineer for approval prior to welding. For ASTM A252 material, mill certification for each lot of pipe to be welded shall accompany the submittal.

Weld splicing of steel casings for cast-in-place concrete piles shall be the Contractor's responsibility. Casings that collapse or are not watertight, shall be replaced at the Contractor's expense.

Steel casing joints shall not be offset more than $^1/_{16}$-inch.

6-05.3(7) Storage and Handling

The Contractor shall store and handle piles in ways that protect them from damage.

6-05.3(7)A Timber Piles

Timber piling shall be stacked closely and in a manner to prevent warping. The ground beneath and around stored piles shall be cleared of weeds, brush, and rubbish. Piling shall be covered against the weather if the Engineer requires it.

The Contractor shall take special care to avoid breaking the surface of treated piles. They shall be lifted and moved with equipment, tools, and lifting devices which do not penetrate or damage the piles. If timber piles are rafted, any attachments shall be within 3-feet of the butts or tips. Any surface cut or break shall be repaired as per Section 9-09.3. The Engineer may reject any pile because of a cut or break.

6-05.3(7)B Precast Concrete Piles

The Contractor shall not handle any pile until test cylinders made with the same batch of concrete as the pile reach a compressive strength of at least 3,300 psi.

Storing and handling methods shall protect piles from fractures by impact and undue bending stresses. Handling methods shall never stress the reinforcement more than 12,000 psi. An allowance of twice the calculated load shall be made for impact and shock effects. The method of lifting the piles shall be submitted to the Engineer for approval. The Contractor will take extra care to avoid damaging the surface of any pile to be used in seawater or alkaline soil.

6-05.3(7)C Steel Casings and Steel Piles

The Engineer will reject bent, deformed, or kinked piles that cannot be straightened without damaging the metal.

6-05.3(8) Pile Tips and Shoes

The Contracting Agency prefers that timber piles be driven with squared ends. But if conditions require, they may be shod with metal shoes. Pile tips and shoes shall be securely attached to the piles in accordance with the manufacturer's recommendations.

Where called for in the Contract, conical steel pile tips shall be used when driving steel casings. The tips shall be inside fit, flush-mounted such that the tip and/or weld bead does not protrude more than $^{1}/_{16}$-inch beyond the nominal outside diameter of the steel casing.

If conical tips are not specified, the lower end of each casing shall have a steel driving plate that is thick enough to keep the casing watertight and free from distortion as it is driven. The diameter of the steel driving plate shall not be greater than the outside diameter of the steel casing.

Where called for in the Contract, inside-fit cutting shoes shall be used when driving open-ended steel piles. The cutting shoes shall be flush-mounted such that the shoe and/or weld bead does not protrude more than $^{1}/_{16}$-inch beyond the nominal outside diameter of the steel pile. The cutting shoe shall be of an inside diameter at least $^{3}/_{4}$-inch less than the nominal inside diameter of the steel pile.

Pile tips or shoes shall be of a type denoted in the Qualified Products List. If pile tips or shoes other than those denoted in the Qualified Products List are proposed, the Contractor shall submit shop drawings of the proposed pile tip along with design calculations, specifications, material chemistry and installation requirements, to the Engineer for approval. The Contractor shall also submit evidence of a pile driving test demonstrating suitability of the proposed pile tip. The test shall be performed in the presence of the Engineer or an acceptable independent testing agency. The test shall consist of driving a pile fitted with the proposed tip. If the pile cannot be visually inspected (see Section 6-05.3(11)F), a sacrificial pile fitted with the proposed tip shall be driven outside the proposed foundation limits. The pile shall be driven to a depth sufficient to develop the required ultimate bearing capacity as called for in the Contract, in ground conditions determined to be equivalent to the ground conditions at the project site. For closed-ended casings or piles, the pile need not be removed if, in the opinion of the Engineer, the pile can be inspected for evidence of damage to the pile or the tip. For open-ended steel casings or piles, timber piles or H-piles, the pile shall be removed for inspection.

6-05.3(9) Pile Driving Equipment

6-05.3(9)A Pile Driving Equipment Approval

Prior to driving any piles, the Contractor shall submit to the Engineer for approval the details of each proposed pile driving system. The pile driving system shall meet the minimum requirements for the various combinations of hammer type and pile type specified in this Section. These requirements are minimums and may need to be increased in order to ensure that the required ultimate bearing capacity can be achieved, that minimum tip elevations can be reached, and to prevent pile damage.

The Contractor shall submit a wave equation analysis for all pile driving systems used to drive piling with required ultimate bearing capacities of greater than 300 tons. The wave equation analysis shall be performed by, and bear the stamp of, a civil engineer licensed in the State of Washington. The wave equation analysis shall be performed in accordance with the requirements of this section and the user's manual for the program. The wave equation analysis shall verify that the pile driving system proposed does not produce stresses greater than 50,000 psi or 90 percent of the yield stress whichever is less, for steel piles, or steel casings for cast-in-place concrete piles.For prestressed concrete piles, the allowable driving stress shall be 3 $\sqrt{f'cm}$ plus prestress in tension, and 0.85f_c' minus prestress in compression. For precast concrete piles that are not prestressed, the allowable driving stress shall be 70 percent of the yield stress of the steel reinforcement in tension, and 0. 85f_c' in compression. The wave equation shall also verify that the pile driving system does not exceed the refusal criteria at the depth of penetration anticipated for achieving the required ultimate bearing capacity and minimum tip elevation. Furthermore, the wave equation analysis shall verify that at the maximum driving resistance specified in the Contract, the driving resistance is 100 blows per foot or less. Unless otherwise specified in the Contract, or directed by the Engineer, the following default values shall be used as input to the wave equation analysis program:

Output option (IOUT)	0
Factor of safety applied to (R_{ult})	1.0
Type of damping	Smith
Residual stress option	No

R_{ult} is the resistance of the pile used in the wave equation analyses. If the ultimate bearing capacity equals the maximum driving resistance, a setup factor of 1.3 may be used in the wave equation analysis to account for pile setup. To use a setup factor in the wave equation analysis, Rult in the analysis is the ultimate bearing capacity divided by 1.3. If the maximum driving resistance exceeds the ultimate bearing capacity, no setup factor should be used, and R_{ult} is equal to the maximum driving resistance of the pile.

Hammer efficiencies:	For Analysis of Driving Resistance	For Analysis of Driving Stresses
Single acting diesel hammers	0.72	0.84
Closed-ended diesel hammers	0.72	0.84
Single acting air/steam hammers	0.60	0.70
Double acting air/steam hammers	0.45	0.53
Hydraulic hammers or other external combustion hammers having ram velocity monitors that may be used to assign an equivalent stroke.	0.85	1.00

Within 15 working days after the Engineer receives the submittal, the Contractor will be notified of the Engineer's acceptance or rejection. If the Contractor wishes to change the pile driving system after the Contractor's proposed system has been approved, the system must be submitted for approval to the Engineer, and up to an additional 10 working days for approval will be required.

6-05.3(9)B Pile Driving Equipment Minimum Requirements

For each drop hammer used, the Contractor shall weigh it in the Engineer's presence or provide the Engineer with a certificate of its weight. The exact weight shall be stamped on the hammer. Drop hammers shall weigh not less than:

1. 3,000 pounds for piles under 50-feet long that have an ultimate bearing capacity of not more than 60 tons, and
2. 4,000 pounds for piles 50-feet and longer or that have an ultimate bearing capacity of 60 to 90 tons.

If a drop hammer is used for timber piles, it is preferable to use a heavy hammer and operate with a short drop.

For each diesel, hydraulic, steam, or air-driven hammer used, the Contractor shall provide the Engineer with the manufacturer's specifications and catalog. These shall show all data needed to calculate the developed energy of the hammer used.

Underwater hammers may be used only with approval of the Engineer.

Drop hammers on timber piles shall have a maximum drop of 10-feet. Drop hammers shall not be used to drive timber piles that have ultimate bearing capacities of more than 60 tons.

When used on timber piles, diesel, hydraulic, steam, or air-driven hammers shall provide at least 13,000 foot-pounds of developed energy per blow. The ram of any diesel hammer shall weigh at least 2,700 pounds.

Precast concrete and precast-prestressed concrete piles shall be driven with a single-acting steam, air, hydraulic, or diesel hammer with a ram weight of at least half as much as the weight of the pile, but never less than the minimums stated below. The ratio of developed hammer energy to ram weight shall not exceed six. Steel casings for cast-in-place concrete, steel pipe, and steel H-piles shall also be driven with diesel, hydraulic, steam, or air hammers. These hammers shall provide at least the following developed energy per blow:

Maximum Driving Resistance (Tons)	Minimum Developed Energy per Blow (ft-lbs) Air or Steam Hammers	Open Ended Diesel Hammers	Closed Ended Diesel Hammers	Hydraulic Hammers
Up to 165	21,500	23,000	30,000	18,500
166 to 210	27,500	29,500	38,000	23,500
211 to 300	39,000	41,500	54,000	33,500
301 to 450	59,000	63,000	81,000	50,500

In addition, the ram of any diesel or hydraulic hammer shall have the following minimum weights:

Maximum Driving Resistance (Tons)	Minimum Ram Weight (lbs)
Up to 165	2,700
166 to 210	4,000
211 to 300	5,000
301 to 450	6,500

These requirements for minimum hammer size may be waived if to the satisfaction of the Engineer a wave equation analysis is performed which demonstrates the ability of the hammer to obtain the required bearing capacity and minimum tip elevation without damage to the pile.

Vibratory hammers may be used to drive piles provided the location and plumbness requirements of this section are met. The required bearing capacity for all piles driven with vibratory hammers will be determined according to 6-05.3(12) by driving the pile at least an additional 2-feet using an impact hammer. This method of determining bearing capacity will be accepted provided the blows per inch are either constant or increasing. If the pile cannot be driven 2-feet, the pile will be considered acceptable for bearing if the pile is driven to refusal.

If water jets are used, the number of jets and water volume and pressure shall be enough to erode the material next to the pile at the tip. The equipment shall include a minimum of two water-jet pipes and two $^{3}/_{4}$-inch jet nozzles. The pump shall produce a constant pressure of at least 100 psi at each nozzle.

6-05.3(9)C Pile Driving Leads

All piles shall be driven with fixed-lead drivers. The leads shall be fixed on the top and bottom during the pile driving operation. Leads shall be long enough to eliminate the need for any follower (except for timber piles as specified in Section 6-05.3(11)E). To avoid bruising or breaking the surface of treated timber piles, the Contractor shall use spuds and chocks as little as possible. In building a trestle or foundation with inclined piles, leads shall be adapted for driving batter piles.

A helmet of the right size for the hammer shall distribute the blow and protect the top of steel piling or casings from driving damage. The helmet shall be positioned symmetrically below the hammer's striking parts, so that the impact forces are applied concentric to the pile top.

Pile driving leads other than those fixed at the top and bottom may be used to complete driving, if approved by the Engineer, when all of the following criteria are met:

1. Each plumb and battered pile is located and initially driven at least 20-feet in true alignment using fixed leads or other approved means.
2. The pile driving system (hammer, cushion and pile) will be analyzed by Pile Driving Analyzer (PDA) to verify driving stresses in the pile are not increased due to eccentric loading during driving, and transferred hammer energy is not reduced due to eccentric loading during driving, for all test piles and at least one production pile per pier. Unless otherwise specified, the cost of PDA testing shall be incidental to the various unit contract prices for driving piles.

6-05.3(10) Test Piles

If the Contract or the Engineer call for it, the Contractor shall drive test piles to determine pile lengths required to reach the required ultimate bearing capacity, penetration, or both. Test piles shall be:

1. Made of the same material and have the same tip diameter as the permanent piles (although test piles for treated timber piles may be either treated or untreated),
2. Driven with pile tips if the permanent piles will have tips,
3. Prebored when preboring is specified for the permanent piles,
4. Identical in cross-section and other characteristics to the permanent piles when the test piles are steel casings for cast-in-place concrete piles, precast concrete, precast-prestressed concrete or steel pipe or H-pile,
5. Long enough to accommodate any soil condition,
6. Driven with equipment and methods identical to those to be used for the permanent piles,
7. Located as the Engineer directs, and
8. Driven before permanent piles in a given pier.

Test piles may also be driven by the Contractor, (at no cost to the Contracting Agency,) as evidence that the pile driving system selected will not damage the pile or result in refusal prior to reaching any specified minimum tip elevation.

Timber test piles shall be driven outside the footing and cut off 1 foot below the finished ground line. Timber test piles shall not be used in place of permanent piles.

Steel and all types of concrete test piles shall become permanent piles. The Contracting Agency has reduced the number of permanent piles by the number of test piles.

The Contractor shall base test pile length on test-hole data in the contract. Any test piles that prove to be too short shall be replaced (or spliced if the Contract allows splicing) at the Contractor's expense.

In foundations and trestles, test piles shall be driven to at least 15 percent more than the ultimate bearing capacity required for the permanent piles, except where pile driving criteria is determined by the wave equation. When pile driving criteria is specified to be determined by the wave equation, the test piles shall be driven to the same ultimate bearing capacity as the production piles. Test piles shall penetrate at least to any minimum tip elevation specified in the Contract. If no minimum tip elevation is specified, test piles shall extend at least 10-feet below the bottom of the concrete footing or ground line, and 15-feet below the bottom of the concrete seal.

When any test pile to be left as a permanent pile has been so damaged by handling or driving that the Engineer believes it unfit for use, the Contractor shall remove and replace the pile at no additional cost to the Contracting Agency. The Engineer may direct the Contractor to overdrive the test pile to more than 15 percent above the ultimate bearing capacity for permanent piles, or if the wave equation is used to determine driving criteria, the Engineer may direct the Contractor to overdrive the test pile above the ultimate bearing capacity. In these cases, the overdriving shall be at the Contractor's expense. But if pile damage results from this overdriving, any removal and replacement will be at the Contracting Agency's expense.

6-05.3(11) Driving Piles

6-05.3(11)A Tolerances

For elevated pier caps, the tops of piles at cut-off elevation shall be within 2-inches of the locations indicated in the Contract. For piles capped below final grade, the tops of piles at cut-off elevation shall be within 6-inches of the horizontal locations indicated in the Contract. No pile edge shall be nearer than 4-inches from the edge of any footing or cap. Piles shall be installed such that the axial alignment of the top 10-feet of the pile is within 4 percent of the specified alignment. No misaligned steel or concrete piles shall be pulled laterally. A properly aligned section shall not be spliced onto a misaligned section for any type of pile. Unless the Contract shows otherwise, all piles shall be driven vertically.

6-05.3(11)B Foundation Pit Preparation

The Contractor shall replace (and bear the cost of replacing) any pile damaged or destroyed before or during driving.

The Contractor shall completely dig all foundation pits (and build any required cofferdams or cribs) before driving foundation piles. The Contractor shall adjust pit depths to allow for upheaval caused by pile-driving, judging the amount of adjustment by the nature of the soil. Before constructing the footing or pile cap, the Contractor shall restore the pit bottom to correct elevation by removing material or by backfilling with granular material.

6-05.3(11)C Preparation for Driving

Treated and untreated timber piles shall be freshly cut square on the butt ends just before they are driven. If piles will be driven into hard material, caps, collars, or bands shall be placed on the butt ends to prevent crushing or brooming. If the head area of the pile is larger than that of the hammer face, the head shall be snipped or chamfered to fit the hammer. On treated piles, the heads shall be snipped or chamfered to at least the depth of the sapwood to avoid splitting the sapwood from the pile body.

The Contractor shall match timber pile sizes in any single bent to prevent sway braces from undue bending or distorting.

When driven, pile faces shall be turned as shown in the Plans or as the Engineer directs.

No precast-prestressed pile shall be driven until test cylinders poured with it reach at least the specified compressive strength shown in the Contract. On all other precast piles, the cylinders must reach a compressive strength of at least 4,000 psi before the piles are driven.

Helmets of approved design shall protect the heads of all precast concrete piles as they are driven. Each helmet shall have fitted into it a cushion next to the pile head. The bottom side of the helmet shall be recessed sufficiently to accommodate the required pile cushion and hold the pile in place during positioning and driving. The inside helmet diameter shall be determined before casting the pile, and the head of the pile shall be formed to fit loosely inside the helmet.

Steel Casing, steel pipe or H-piles shall have square-cut ends.

6-05.3(11)D Achieving Minimum Tip Elevation and Bearing

Once pile driving has started, each pile shall be driven continuously until the required ultimate bearing capacity shown in the contract has been achieved. Pauses during pile driving, except for splicing, mechanical breakdown, or other unforeseen events, shall not be allowed.

If the Contract specifies a minimum tip elevation, the pile shall be driven to at least the minimum tip elevation, even if the ultimate bearing capacity has been achieved, unless the Engineer directs otherwise. If a pile does not develop the required ultimate bearing capacity at the minimum tip elevation, the Contractor shall continue driving the pile until the required bearing capacity is achieved. If no minimum tip elevation is specified, then the piles shall be driven to the ultimate bearing capacity shown in the Contract and the following minimum penetrations:

Pile supporting cross-beams, bents, elevated pile caps elevation	10-feet below final top of ground
Piles supporting foundations	10-feet below bottom of foundation
Piles with a concrete seal	15-feet below bottom of seal

If overdriving is required in order to reach a specified minimum tip elevation, the Contractor shall provide a pile driving system which will not result in damage to the pile or refusal before the minimum tip elevation is reached. The cost of overdriving shall be incidental to the various unit contract prices for furnishing and driving piles.

So long as the pile is not damaged and the embankment or foundation material being driven through is not permanently damaged, the Contractor shall use normal means necessary to:

1. Secure the minimum depth specified,
2. Penetrate hard material that lies under a soft upper layer,
3. Penetrate through hard material to obtain the specified minimum tip elevation, or
4. Penetrate through a previously placed embankment.

Normal means refer to methods such as preboring, spudding, or jetting piles. Blasting or drilling through obstructions are not considered normal means.

Prebored holes and pile spuds shall have a diameter no larger than the least outside dimension of the pile. After the pile is driven, the Contractor shall fill all open spaces between the pile and the soil caused by the preboring or spudding with dry sand, or pea gravel, or controlled density fill as approved by the Engineer.

If water jets are used, the jets shall be withdrawn before the pile reaches its final penetration, and the pile shall then be driven to its final penetration and ultimate bearing capacity. The pile shall be driven a minimum of 2-feet to obtain the ultimate bearing capacity after the jets are withdrawn, or to refusal, whichever occurs first. If the water jets loosen a pile previously driven, it shall be redriven in place or pulled and replaced by a new pile. To check on pile loosening, the Contractor shall attempt to redrive at least one in every five piles, but no less than one pile per bent or pier.

The various unit contract prices for driving piles shall cover all costs related to the use of water jets, preboring, or spudding. The Contracting Agency will not pay any costs the Contractor incurs in redriving piles loosened as a result of using water jets, preboring, or spudding.

If the Engineer requires, the Contractor shall overdrive the pile beyond the ultimate bearing capacity and minimum tip elevation shown in the Contract. In this case, the Contractor will not be required to:

1. Use other than normal means to achieve the additional penetration;
2. Bear the expense of removing or replacing any pile damaged by overdriving; or
3. Bear the expense of overdriving the pile more than 3-feet as specified in Section 6-05.5.

In driving piles for footings with seals, the Contractor shall use no method (such as jetting or preboring) that might reduce friction capacity.

6-05.3(11)E Use of Followers for Driving

Followers shall not be used to drive concrete or steel piles. On timber piles, the Contractor may use steel (not wooden) followers if the follower fits snugly over the pile head. If a follower is used, the Contractor shall, in every group of 10 piles, drive one long pile without a follower, but no less than one pile per bent or pier, to the required ultimate bearing capacity and minimum tip elevation. This long pile shall be used to test the bearing capacity of the piles driven with a follower in the group. The tip elevation of the long pile shall be similar to the elevation of the piles driven with the follower. If the tip elevations are significantly different, as determined by the Engineer, the Contractor shall redrive the remaining piles in the group to the tip elevation of the longer pile.

6-05.3(11)F Pile Damage

The Contractor shall remove and replace (and bear the cost of doing so) any pile that is damaged as determined by the Engineer.

After driving a steel casing for a cast-in-place concrete pile, the Contractor shall leave it empty until the Engineer has inspected and approved it. The Contractor shall make available to the Engineer a light suitable for inspecting the entire length of its interior. The Engineer will reject any casing that is improperly driven, that shows partial collapse that would reduce its ultimate bearing capacity, or that has been reduced in diameter, or that will not keep out water. The Contractor shall replace (and bear the cost of replacing) any rejected casing.

Pile heads which have been broomed, rolled, or otherwise significantly damaged as determined by the Engineer shall be cut back to undamaged material before proceeding with driving as well as final acceptance of the pile.

6-05.3(11)G Pile Cutoff

The Contractor shall trim the tops of all piles to the true plane shown in the Contract and to the elevation the Engineer requires. If a pile is driven below cutoff elevation without the Engineer's approval, the Contractor shall remove and replace it (and bear the costs of doing so), even if this requires a longer pile. Any pile that rises as nearby piles are driven, shall be driven down again if the Engineer requires.

Any piles under timber caps or grillages shall be sawed to the exact plane of the structure above them and fit it exactly. No shimming on top of timber piles to adjust for inaccurate pile top elevations will be permitted. If a timber pile is driven out of line, it shall be straightened without damage before it is cut off or braced.

Steel casing shall be cut off at least 6-inches below the finished ground line or at the low water line if the casing will be visible as determined by the Engineer.

6-05.3(11)H Pile Driving From or Near Adjacent Structures

The Contractor shall not drive piling from an existing structure unless all of the following conditions are met:

1. The existing structure will be demolished within the contract.
2. The existing structure is permanently closed to traffic, and
3. Working drawings are submitted in accordance with Sections 6-01.9 and 6-02.3(16), showing the structural adequacy of the existing structure to safely support all of the construction loads.

To minimize the detrimental effects of pile driving vibrations on new concrete less than 28 days old, piles shall not be driven closer to the new concrete than the distance determined from the following formula:

D = C times the square root of E

Where: D = distance in feet

E = rated hammer energy in foot-pounds

C = coefficient shown below based on the number ofdays of curing time

Curing Time (days)	Coefficient (C)	Curing Time (days)	Coefficient (C)
1	0.34	6	0.12
2	0.23	7-9	0.11
3	0.18	10-13	0.10
4	0.15	14-20	0.09
5	0.13	21-28	0.08

This distance may be reduced if approved in writing by the Engineer.

6-05.3(12) Determination of Bearing Values

The following formula shall be used to determine ultimate bearing capacities:

$P = F \times E \times Ln(10N)$

Where: P = ultimate bearing resistance, in tons

F = 1.8 for air/steam hammers

= 1.2 for open ended diesel hammers and precast concrete piles

= 1.6 for open ended diesel hammers and steel or timber piles

= 1.2 for closed ended diesel hammers

= 1.9 for hydraulic hammers

= 0.9 for drop hammers

E = developed energy, equal to W times H[1], in ft-kips

W = weight of ram, in kips

H = vertical drop of hammer or stroke of ram, in feet

N = average penetration resistance in blows per inch for the last 4-inches of driving

Ln = the natural logarithm, in base "e"

[1]For closed-end diesel hammers (double-acting), the developed hammer energy (E) is to be determined from the bounce chamber reading. Hammer manufacturer calibration data may be used to correlate bounce chamber pressure to developed hammer energy. For double acting hammer hydraulic and air/steam hammers, the developed hammer energy shall be calculated from ram impact velocity measurements or other means approved by the Engineer. For open ended diesel hammers (single-acting) use the blows per minute to determine the developed energy (E).

The above formula applies only when:

1. The hammer is in good condition and operating in a satisfactory manner;
2. A follower is not used;
3. The pile top is not damaged;
4. The pile head is free from broomed or crushed wood fiber;
5. The penetration occurs at a reasonably quick, uniform rate; and the pile has been driven at least 2-feet after any interruption in driving greater than 1 hour in length.
6. There is no perceptible bounce after the blow. If a significant bounce cannot be avoided, twice the height of the bounce shall be deducted from "H" to determine its true value in the formula.
7. For timber piles, bearing capacities calculated by the formula above shall be considered effective only when it is less than the crushing strength of the piles.
8. If "N" is greater than or equal to 1.0 blow/inch.

If "N" required to achieve the required ultimate bearing capacity using the above formula is less than 1.0 blow/inch, the pile shall be driven until the penetration resistance is a minimum of 1.0 blow/inch for the last 2-feet of driving.

The Engineer may require the Contractor to install a pressure gauge on the inboard end of the hose to check pressure at the hammer.

If water jets are used in driving, bearing capacities shall be determined either: (1) by calculating it with the driving data and the formula above after the jets have been withdrawn and the pile is driven at least 2-feet, or (2) by applying a test load.

6-05.3(13) Treatment of Timber Pile Heads

After cutting timber piles to correct elevation, the Contractor shall thoroughly coat the heads of all untreated piles with two coats of an approved preservative that meets the requirements of Section 9-09 (except concrete-encased piles).

After cutting treated timber piles to correct elevation, the Contractor shall brush three coats of an approved preservative that meets the requirements of Section 9-09 on all pile heads (except those to be covered with concrete footings or concrete caps). The pile heads shall then be capped with alternate layers of an approved roofing asphalt and a waterproofing fabric that conforms to Section 9-11.2. The cap shall be made of four layers of an approved roofing asphalt and three layers of fabric. The fabric shall be cut large enough to cover the pile top and fold down at least 6-inches along all sides of the pile. After the fabric cover is bent down over the pile, its edges shall be fastened with large-head galvanized nails or with three turns of galvanized wire. The edges of the cover shall be neatly trimmed.

On any treated timber pile encased in concrete, the cut end shall receive two coats of an approved preservative that meets the requirements of Section 9-09 and then a heavy coat of an approved roofing asphalt.

6-05.3(14) Extensions and Build-ups of Precast Concrete Piles

The Contractor shall add extensions, or build-ups (if necessary) on precast concrete piles after they are driven to the required ultimate bearing capacity and minimum tip elevation.

Before adding extensions or build-ups to precast-prestressed piles, the Contractor shall remove any spalled concrete, leaving the pile fresh-headed and with a top surface perpendicular to the axis of the pile. The concrete in the build-up shall be Class 5000.

Before adding to non-prestressed precast concrete piles, the Contractor shall cut the pile head away to a depth 40 times the diameter of the vertical reinforcing bar. The final cut shall be perpendicular to the axis of the pile. Reinforcement of the same density and configuration as used in the pile shall be used in the build-up and shall be fastened firmly to the projecting steel. Forms shall be placed to prevent concrete from leaking along the pile. The concrete in the build-up shall be Class 4000.

Just before placing the concrete for extensions or build-ups to precast or precast-prestressed concrete piles, the Contractor shall thoroughly wet the top of the pile. Forms shall remain in place at least three days.

6-05.3(15) Completion of Cast-In-Place Concrete Piles

After approval by the Engineer, driven casings shall be cut off horizontally at the required elevation. They shall be clean and free of water when concrete and reinforcing steel are placed.

These piles shall consist of steel casings driven into the ground, reinforced as specified, and filled with Class 4000P concrete.

6-05.3(15)A Reinforcement

All bars shall be fastened rigidly into a single unit, then lowered into the casing before the concrete is placed. Loose bars shall not be used.

Spiral hooping reinforcement shall be deformed steel bar, plain steel bar, cold-drawn wire, or deformed wire.

6-05.3(15)B Placing Concrete

Before placing concrete, the Contractor shall remove all debris and water from the casing. If the water cannot be removed, the casing shall be removed (or cut off 2-feet below the ground and filled with sand) and a new one driven.

The Contractor shall place concrete continuously through a 5-foot rigid conduit directing the concrete down the center of the pile casing, ensuring that every part of the pile is filled and the concrete is worked around the reinforcement. The top 5-feet of concrete shall be placed with the tip of the conduit below the top of fresh concrete. The Contractor shall vibrate, as a minimum, the top 10-feet of concrete. In all cases, the concrete shall be vibrated to a point at least 5-feet below the original ground line.

6-05.4 Measurement

Measurement for driving (type) pile will be the number of piles driven in place.

In these categories, measurement will be the number of linear feet driven below cutoff or as shown in the Engineer's order list:

1. Furnishing timber piling (untreated or name of treatment).
2. Precast concrete and precast-prestressed concrete piling.

In these categories, measurement will be the number of linear feet driven below cutoff, but no Engineer's order list will be provided:

1. Cast-in-place concrete piling.
2. Furnishing steel piling.

Measurement for furnishing and driving test piles will be the number actually furnished and driven as the Contract requires.

Measurement for steel pile tips or shoes will be by the number of tips or shoes actually installed and driven in place on steel casings or steel piles.

6-05.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

"Furnishing and Driving (type) Test Pile", per each.

The unit contract price per each for "Furnishing and Driving (type) Test Pile" shall be full pay for furnishing and driving test piles to the ultimate bearing capacity or penetration required by the Engineer, furnishing and installing a pile tip when pile tips are specified for the permanent piles, preboring when preboring is specified for the permanent piles, for pulling the piles or cutting them off as required, and for removing them from the site or for delivery to the Contracting Agency for salvage when ordered by the Engineer. This price shall also include all costs in connection with moving all pile driving equipment or other necessary equipment to the site of the work and for removing all such equipment from the site after the piles have been driven. If, after the test piles have been driven, it is found necessary to eliminate the piling from all or any part of the structure, no additional pay will be allowed for moving the pile driving equipment to and from the site of the work.

"Driving Timber Pile (untreated or name treatment)", per each.

The unit contract price per each for "Driving Timber (type) Pile" shall include any metal shoes which the Contractor has determined to be beneficial to the pile driving.

"Driving Conc. Pile (size)", per each.

"Driving St. Pile", per each.

The unit contract price per each for "Driving (type) Pile (____)" shall be full pay for driving the pile to the ultimate bearing and/or penetration specified. When overdriving piles beyond the ultimate bearing capacity and minimum tip elevation specified in the Contract is required by the Engineer, payment for the first 3-feet of overdriving will be included in the unit contract price for "Driving (type) Pile". Additional penetration beyond the first 3-feet of overdriving will be paid for on the basis of force account work as covered in Section 1-09.6.

"Furnishing Timber Piling (untreated or name treatment)", per linear foot.

"Furnishing Conc. Piling (size)", per linear foot.

"Furnishing St. Piling", per linear foot.

The unit contract price per linear foot for "Furnishing (type) Piling (____)" shall be full pay for furnishing the piling specified. Such price shall also be full pay, when measurement includes, for piling length ordered but not driven.

Precast Concrete Pile Buildup", by force account.

Payment for buildups of precast or precast-prestressed concrete piles will be made on the basis of force account work as covered in Section 1-09.6. No payment will be made for build-ups or additional lengths of build-up made necessary because of damage to the piling during driving. The length of splice for precast concrete piles includes the length cut off to expose reinforcing steel for the splice. The length of splice for precast-prestressed piles includes the length in which holes are drilled and reinforcing bars are grouted.

For the purpose of providing a common proposal for all bidders, the Contracting Agency entered an amount for "Precast Concrete Pile Buildup" in the proposal to become part of the total bid by the Contractor.

"Furnishing Steel Pile Tip or Shoe (size)", per each.

6-06 BRIDGE RAILINGS

6-06.1 Description

This work consists of providing and building bridge railings that meet the requirements of the Plans, these Specifications, and the Engineer.

6-06.2 Materials

Materials shall meet the requirements of the following sections:

Timber Railing	9-09
Metal Railing	9-06.18

6-06.3 Construction Requirements

6-06.3(1) Timber Railings

Wheel guards and railings shall be true to line and grade and framed accurately. The Contractor shall follow Section 6-04 whenever this subsection does not specify a construction method.

Unless the Plans show otherwise, wheel guards shall be:

1. Beveled and surfaced on the roadway side and surfaced on the top edge. They may be surfaced on four sides (S4S).
2. Laid in sections at least 12-feet long.
3. Bolted through the floor plank and outside stringer (or nailing piece) with $^{3}/_{4}$-inch diameter bolts spaced no more than 4-feet apart.

All rails and rail post material shall be S4S and painted as required in Section 6-07. Railing members shall be fastened securely together, with the bolts tightened once at installation and again just before the Contracting Agency's final acceptance of the contract.

6-06.3(2) Metal Railings

Metal railing includes posts, web members, and horizontal members of the sidewalk and roadway railing. Unless the Plans or Special Provisions show otherwise, these shall be made of aluminum alloy or steel.

Before fabricating the railing, the Contractor shall submit six copies of the shop plans for the Engineer's approval. The Contractor may substitute other rail connection details for those shown in the Plans if details of these changes show in the shop plans and if the Engineer approves. In approving shop plans, the Engineer indicates only that they are adequate and complete enough. Approval does not indicate a check on dimensions.

Anchor bolts or wedge anchors shall be positioned with a template to ensure that bolts match the hole spacing of the bottom channels or anchorage plates.

Where specified, cover plates shall fit the bottom channel tightly after being snapped into position.

Metal railings shall be installed true to line and grade (or camber). After first setting the railing, the Contractor shall readjust all or part of it, if necessary, to create an overall line and grade pleasing to the eye.

6-06.4 Measurement

Timber railing will be measured by the thousand board feet (MBM) as shown in Section 6-04.

Metal railing will be measured by the linear foot along the line and slope at the base of the completed railing.

6-06.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

"Timber and Lumber (untreated or name treatment)", per MBM.

"Bridge Railing Type .____", per linear foot.

In case no item is included in the contract for "Bridge Railing Type ____" and payment is not otherwise provided, all metal railings shall be included in the lump sum contract price for "Structural Carbon St." as specified in Section 6-03.

6-07 PAINTING

6-07.1 Description

This work consists of surface preparation, containing, testing and disposing of surface preparation debris, furnishing and applying paint, shielding adjacent areas from unwanted paint, and cleaning up after painting is completed. The work shall comply with all requirements of the Plans, these Specifications, and the Engineer.

6-07.2 Materials

Paint materials shall comply with the requirements in Section 9-08.

Material used for field abrasive blasting shall meet Military Specification MIL-A-22262A(SH) as listed on QPL-22262-15 as maintained by the Department of the Navy. The Contractor shall provide the Engineer with certified test results from the abrasive blast media manufacturer showing that the abrasive blast material meets the Military Specification. In addition, the Contractor shall blend an additive with the abrasive blast media that renders the blast residual to a non-hazardous waste condition.

6-07.3 Construction Requirements

6-07.3(1) Painting New Steel Structures

All material classified as structural steel shall be painted with a shop applied, inorganic zinc silicate primer, followed by a field applied two coat paint system after field erection, cleaning, and spot priming have been completed. Except as otherwise specified, all steel surfaces shall be painted with three coats of paint. Steel surfaces embedded in concrete and faying (contact) surfaces of bolted connections (including all surfaces internal to the connection and all filler plates) shall receive the prime coat only. Stainless steel surfaces shall not be painted. Galvanized surfaces shall not be painted unless specified in the Plans or Special Provisions. Painting of galvanized surfaces, if so specified, shall be in accordance with Section 6-07.3(4).

The painting system shall consist of three coats as follows:

	Method A	Method B
Primer Coat	inorganic zinc	Inorganic zinc
2nd Coat	epoxy	B-11-99 Field applied
3rd Coat	aliphatic urethane	C-11-99 Field applied

Once a paint system has been selected, that system shall be used throughout the structure.

Terminology used herein is in accordance with the definitions used in Volume 2, Systems and Specifications of the SSPC Steel Structures Painting Manual, current edition.

Prior to any coating materials being utilized, the Contractor shall submit the product data sheets to the Engineer for approval. The product data sheets shall include all application instructions including the mixing and thinning directions, the recommended spray nozzles and pressures, the minimum and maximum drying time between coats, friction coefficient of the faying surface, restrictions on temperature and humidity, and the repair procedures. In addition, the Contractor shall submit to the Engineer for approval an abrasive blast procedure. The procedure shall include the type of equipment and abrasive media to be used.

Paint formulations to be used on faying surfaces shall be Class B coatings with a mean slip coefficient not less than 0.50. The slip coefficient shall be determined by testing in accordance with “Test Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints” as adopted by the Research Council on Structural Connections. Test results and the paint manufacturer’s Certificate of Compliance shall be submitted to the Engineer for approval with the structural steel shop drawings.

For contracts in which more than 20,000 pounds of steel are to be painted, the manufacturer of the paint system shall have a technical representative present at the job site for the first day of painting. After the first day of painting the technical representative shall remain available for contact in the event of technical difficulties in applying the paint system.

During fabrication and shop painting, the Contractor shall provide access meeting the approval of the Engineer to permit inspection of the steel. The access shall not mar or damage any freshly painted surfaces.

The Contractor shall select a primer from one of the approved products listed in the Qualified Products List. The field applied primer, the second coat and the third coat shall all be selected from the same manufacturer from one of the approved coating systems listed in the Qualified Products List.

The color for the second coat shall be a contrasting color to the third coat. The color for the third coat shall be as specified in the Special Provisions.

Steel surfaces shall be:

For Method A:

1. Greater than 45ºF and at least 5ºF above the dew point, and
2. Less than 115ºF.

For Method B:

1. Greater than 35ºF and
2. Less than 115ºF

6-07.3(1)A Preparation for Shop Coating

A roughened surface profile shall be provided by an abrasive blasting procedure as approved by the Engineer. The profile shall be one mil minimum or per the paint manufacturer’s recommendation, whichever is greater. The steel surfaces shall be cleaned to a near white condition as per SSPC-SP10.

After being thoroughly cleaned by abrasive blasting as specified above, all structural steel shall be primed within the same working day on which abrasive blasting takes place, and before any rust forms, by spraying with a full coat of inorganic zinc silicate paint. The Contractor shall not begin painting until receiving the Engineer’s approval of the prepared surface. High strength field bolts need not be painted before erection.

Care shall be taken to protect freshly coated surfaces from subsequent abrasive blast cleaning operations. Primed surfaces which are damaged by abrasive blasting shall be thoroughly wire brushed or, if visible rust occurs, re-blasted to a near-white (SSPC-SP10) condition. The wire brushed or abrasive blast cleaned surfaces shall be vacuumed and re-primed by spraying.

6-07.3(1)B Mixing and Thinning the Shop and Field Coatings

The coating shall be mixed with a high shear mixer in accordance with the manufacturer's written recommendations to a smooth, lump-free consistency. Paddle mixers or paint shakers are not allowed. Mixing shall be done, to the extent possible, in the original containers and shall be continued until all of the metallic powder or pigment is in suspension. The mixed coating shall be kept under continuous agitation up to and during the time of application.

In general, the coatings are supplied for use without requiring thinning. If it is necessary to thin the coating for proper application in cool weather, or to obtain better coverage of the urethane coat, the thinning shall be done in accordance with the manufacturer's written recommendations.

6-07.3(1)C Applying The Shop Coating

After the surface to be coated has been cleaned, and has received the Engineer's approval, the primer coat shall be applied so as to produce a uniform, even coating that has fully bonded with the metal.

The coatings shall be applied with the spray nozzles and pressures recommended by the manufacturer of the paint system, so as to attain the film thicknesses specified.

The top surfaces of the top flanges of the steel girders shall not be primed until the welded shear connectors are placed, unless the welded shear connectors are to be placed in the field. Welded shear connectors are not required to be painted except for the weld area.

If the welded shear connectors are to be placed in the field, the area to be welded shall be cleaned of primer by abrasive grinding just prior to welding. After welding, the ground area and the weld shall be cleaned and primed. Surfaces which are inaccessible for painting after erection shall be painted with the two field coats of paint before erection.

Dry film thickness measurements will be made in accordance with Section 6-07.3(5).

6-07.3(1)D Field Coating After Erection

When the erection work has been completed, including all connections and the straightening of any bent metal, all steel surfaces and bolts shall be prepared for painting. All adhering scale, dirt, grease, form oil, or other foreign matter shall be removed by appropriate means and all rusted or uncoated areas including the bolts, nuts, washers and splice plates shall be abrasive blasted to a near-white (SSPC-SP10) condition. All uncoated areas shall be field primed with an organic zinc paint coating selected from the same approved coating system and paint manufacturer as the other coatings for the structure.

After all field priming has been completed the surfaces shall be prepared to receive the final two field coats. The intermediate coat shall be mixed and applied per the manufacturer's written recommendations. The top coat shall also be mixed and applied per the manufacturer's written recommendations. The minimum drying time between coats shall be as shown in the approved product data sheets, but not less than 12 hours. Depending on site conditions, additional time may be required for proper curing before applying succeeding coats. The Contractor shall determine if the coating has cured sufficiently for proper application of succeeding coats. The maximum time between coats shall be in accordance with the manufacturer's written recommendations. If the maximum time between coats is exceeded, all newly coated surfaces shall be completely blast-cleaned again to a near white finish (SSPC-SP10) and re-coated at no additional cost to the Contracting Agency.

Dry film thickness measurements will be made in accordance with Section 6-07.3(5).

Temporary attachments or supports for scaffolding or forms shall not damage the coating system. All paint damage that occurs shall be repaired in accordance with the manufacturer's written recommendations and as follows. On bare areas or areas of insufficient primer thickness, the repair shall include the application of the field applied organic zinc primer system, and the final two coats of the Method A or Method B paint system. On areas where the primer is at least equal to the minimum required dry film thickness, the repair shall include the application of the final two coats of the Method A or Method B paint system. If any abrasive blast cleaning is required in the field it shall be done using an abrasive conforming to Section 6-07.2.

6-07.3(2) Repainting Existing Steel Structures

Unless otherwise provided, maintenance painting includes cleaning and painting all metal surfaces of an existing bridge. These include all metal surfaces that do not touch other metal, wooden floor or truss members, concrete or stone masonry, or other surfaces. Cleaning means removing rust, scale, unsound paint, dirt, grease, and other foreign matter. The Contractor shall clean and paint all exposed metal surfaces that may rust.

The Contractor shall abrasive blast all rust spots in accordance with the SSPC-SP6 Specifications for commercial blast cleaning. The edges of cleaned areas shall show no red or yellow rust. The edges of sound paint shall be feathered smooth. After abrasive blasting, the Contractor shall remove all loose rust, dirt, sand, and dust before painting.

6-07.3(2)A Bridge Cleaning

Bird Guano

Bird guano shall be completely removed prior to any other cleaning. All workers involved with bird guano removal operations shall be protected from absorption, inhalation, or ingestion of bird guano particles by wearing protective clothing as specified in the Contractor's Lead Health Protection Program (LHPP). Bird guano shall be removed in the dry to the extent possible. Following dry removal, the Contractor shall apply a 5.25% sodium hypochlorite solution to the remaining bird guano , followed by hand scrubbing, and pressure flushing as specified. The sodium hypochlorite solution shall not be used as an additive to the water used for pressure flushing, but shall be directly applied onto the areas of remaining bird guano. The bird guano shall be collected in a containment system approved by the Engineer and shall not enter any waterway or the surrounding environment. All bird guano shall be removed and disposed of at a land disposal site approved by the Engineer. The Contractor shall provide the Engineer with one copy of the disposal receipt, which shall include a description of the material disposed of.

Fungicide Treatment

The Contractor shall treat all areas of fungus growth. When treating areas of fungus growth the Contractor shall use special cleaning methods before beginning general surface cleaning operations. The Contractor shall apply a 5.25% sodium hypochlorite solution to the bridge in fungus infested areas for a period recommended by the solution manufacturer or as specified by the Engineer. The sodium hypochlorite solution shall not be used as an additive to the water used for pressure flushing, but shall be directly applied onto the areas of fungus growth.

General Cleaning and Surface Preparation

Following fungicide treatment and removal of the bird guano, all steel surfaces to be painted shall be cleaned by either pressure flushing or sweep blasting. The cleaning process shall remove dirt, loose paint, and other material from the steel surfaces to be painted, but shall not remove well bonded paint. The Contractor shall follow the construction requirements of the cleaning method selected.

Spot abrasive blasting of all rusted steel surfaces and unbonded paint shall follow the pressure flushing or sweep blasting in areas designated by the Engineer. The Contractor shall hand clean, to the satisfaction of the Engineer, all surfaces inaccessible to cleaning with pressure flushing and sweep blasting equipment.

Prior to the application of paint the Contractor shall clean the bridge deck surface for the purpose of dust control.

Pressure Flushing

When pressure flushing is used, it shall be done with clean, fresh water only. No detergents, bleach, or other cleaning agents shall be employed. The pressure flushing equipment shall produce (at the nozzle) at least 3,000 psi with a discharge of at least 4 gpm. The nozzle shall have a 25 degree tip and shall be held no more than 9-inches from the surface being washed. The use of a rotating tip nozzle may be allowed provided:

1. The Contractor requests its use in writing.
2. The pressure equipment shall produce at least 3500 psi at the nozzle.
3. There shall be no additional cost to the Contracting Agency
4. The use of the nozzle has been approved in writing by the Engineer.

The Contractor may pressure flush other portions of the bridge for safety purposes, at no additional expense to the Contracting Agency.

All wash water and debris from pressure flushing shall be filtered through a filter fabric capable of collecting all loose debris and particles. A polypropylene, non-woven, needle-punched geotextile or equivalent shall be used as the filter fabric. The fabric shall have the following properties:

Grab tensile (ASTM D4632):	100 lbs. Min.
Apparent opening size (ASTM D4751):	#70 US Sieve
Permittivity (ASTM D4491):	1.0 sec - 1 or better

The fabric shall be supported underneath the structure to hold the contained material and shall be cleaned at intervals frequent enough to prevent clogging, overflow, or collapse. The debris obtained from the pressure water flushing operation shall be collected and tested in accordance with Section 6-07.3(2)C, and disposed off site at a waste disposal facility approved by the Engineer.

Sweep Blasting and Spot Abrasive Blasting

Sweep or spot abrasive blasting shall not begin until the containment system specified in Section 6-07.3(2)B is in place. No sweep or spot abrasive blasting shall begin until the surfaces are thoroughly dry. The abrasives to be used shall conform to Section 6-07.2. Sweep and spot abrasive blasting shall be done in such a manner that adjacent areas of work that have been partially or entirely completed are protected from damage.

Sweep blasting shall comply with the SSPC-SP 7 requirements. Spot abrasive blasting shall comply with the SSPC-SP 6 requirements.

The abrasive blasters shall be equipped with automatic shutoffs that operate by releasing the trigger mechanism. All abrasive blasting shall be directed towards the bridge center and away from the outboard sides, to facilitate catching all the containment waste. After abrasive blasting, all rust debris, dirt, abrasive and paint residue, and dust shall be completely removed before paint is applied.

6.07.3(2)B Containment of Abrasive Blasting

The Contractor shall protect the surrounding environment from all debris or damage resulting from the Contractor's operation. The Contractor shall take all measures necessary to contain and recover debris generated during cleaning, preparation, and coating operations. The Contractor shall design, construct, and maintain containment systems for abrasive blasting operations in accordance with best management practices. Disposal of the collected materials shall be as specified in the Section 6-07.3(2)C.

1. At the pre-construction conference, the Contractor shall submit a written Containment System Plan, including working drawings as appropriate, describing the methods for waste containment, collection, and disposal, to the Engineer for approval. The Contractor shall prepare and submit the Containment System Plan in accordance with Section 6-01.9. The Contractor shall not begin any abrasive blasting operations until receiving the Engineer's approval of the Containment System Plan.
2. The containment system shall not cause any damage to the existing structure.
3. The Contractor shall enclose all portions of the bridge to be blasted by sweep blasting or spot abrasive blasting as specified. The enclosed area shall consist of that portion below the area to be blasted, and extending up the sides of the structure to above the top of the structure. The enclosed length of each bridge span (defined as pier to pier) shall not exceed one half the length of the span. The containment system may remain open at the top.
4. The containment system shall be capable of being removed rapidly in case of high winds. Abrasive blasting operations shall cease if wind conditions prevent capture of blast rebound and paint residue by the containment system. If there is a question on wind conditions, the Engineer will make the final determination on whether blasting operations shall cease and the containment system removed.
5. The containment system shall not endanger the safety and health of the workers. Access to the containment system shall be designed to prevent any confined materials from escaping.
6. To prevent the weight of the confined materials from causing failure to the containment system, all confined materials shall be collected and secured in sealed containers at the end of each shift daily, at a minimum. No confined materials shall escape during transfer from the containment systems to the sealable containers. All confined materials within the containment system shall be removed and secured in sealable containers prior to relocation or removal of the containment system.
7. If failure to the containment system occurs or if signs of failure to the containment system are present, the Contractor shall stop work immediately. Work shall not resume until the failure has been corrected to the satisfaction of the Engineer.

8. The containment structure shall not be removed and painting operations shall not commence until all abrasive blasted surfaces have been inspected and approved for painting by the Engineer.
9. If the containment structure is removed after the abrasive blasting operation and before the coating operation, the Contractor shall install a drip tarp to prevent spillage of paint onto the waterway and ground surface below.

6-07.3(2)C Testing and Disposal of Containment Waste

Containment waste is defined as all paint chips and debris removed from the steel surface, and all abrasive blast media, as contained by the containment system. After all waste from the containment structures has been collected, the Contractor shall have a minimum of three samples of the wastes tested by an accredited analytical laboratory. Each sample shall be taken from a different storage container unless directed otherwise by the Engineer.

The debris shall be tested for metals using the Toxicity Characteristics Leaching Procedure (TCLP), EPA Methods 1311 and 6010. At a minimum, the materials to be analyzed shall include Arsenic, Barium, Cadmium, Chromium Coppers, Lead, Mercury, Nickel, Selenium, Silver and Zinc.

If the average of the tested samples is at or above all threshold limits as stated in the Dangerous Waste Regulation, Chapter 173-303 WAC, the containment waste will be designated as “Dangerous Waste” and shall be disposed of at a permitted hazardous waste repository. If the average of the tested samples is below the threshold limits, the containment waste will be designated as “Solid Waste” and shall be disposed at a permitted sanitary landfill that will accept the waste. Disposal shall be in accordance with Chapter 173-303 WAC for waste designated “Dangerous Waste” or “Extremely Hazardous Waste” and in accordance with Chapter 173-304 WAC for waste designated as “Solid Waste”.

The Contractor shall supply (2) two copies of the transmittal documents or bill of lading listing the waste material shipped from the construction site to the waste disposal site. One copy of the shipment list shall show the signature of the Engineer and shall have the waste site operator’s confirmation for receipt of the waste.

In the event that the containment wastes are designated as “Dangerous Wastes” or “Extremely Hazardous Waste” under Chapter 173-303 WAC, the Contracting Agency will provide to the Contractor the appropriate EPA identification number.

Unless noted otherwise a waste site will not be provided by the Contracting Agency for the disposal of excess materials and debris.

6-07.3(2)D Drip Tarps

During painting operations the Contractor shall furnish, install, and maintain drip tarps below the areas to be painted to contain all spilled paint, buckets, brushes, and other deleterious material, and prevent such materials from reaching the environment below the bridge. Drip tarps shall be absorbent material and hung to minimize puddling.

The Contractor shall submit to the Engineer for approval, a proposed method for hanging the drip tarps below the paint platforms and connecting them to the bridge, in accordance with Section 6-01.9. After the Contractor has completed painting of the structure, the drip tarps and all connecting hardware shall be removed from the project.

At the pre-construction conference, the Contractor shall submit to the Engineer for approval, a written detailed method for the removal of any accidental spills or drips on traffic that occur during the normal painting operations. A vehicle cleaning station shall be provided.

At the pre-construction conference, the Contractor shall designate, in writing, a supervisory employee of the Contractor who will be on the project at all times and will be fully responsible for taking the required corrective action should any paint damage occur.

6-07.3(2)E Sampling and Testing

The Contractor shall provide the Engineer the following materials and information for testing:

1. One quart of each coating material and of each thinner for testing of each batch or lot that is sampled at the factory at the time of containerizing. The Contracting Agency may, at its discretion, place an Inspector at the site of manufacture.
2. A manufacturer's certificate certifying the test results for each batch of each coat. In addition, if the coating is specified for use on a steel contact surface, the certificate shall certify that the coating material meets the requirements for coefficient of friction.
3. A Product Data Sheet for each coating material and thinner.
4. A Material Safety Data Sheet with the initial sample for each type of coating material and thinner.
5. If the quantity of paint required for each component of the coating system is 20 gallons or less, Item 1 will not apply, and the coating system components will be accepted based on the manufacturer's notarized statement as specified in Section 9-08.3 along with copies of Items 2, 3, and 4.

The following tests will be used to insure that the coating materials meet the requirements of the specifications.

Test	Test Method
Weight-Per-Gallon Determination of Paints and Coatings	ASTM D 1475
Determination of Zinc Dry Films of Paints and Coatings	ASTM D 2371
Coarse Particles in Pigments, Pastes, and Paints	ASTM D 185
Consistency of Paints Using the Stormer Viscometer	ASTM D 562
Fineness of Dispersion of Pigment-Vehicle Systems	ASTM D 1210
Drying, Curing, of Film Formation of Organic Coatings at Room Temperatures	ASTM D 1640
Volatile Content of Paints	ASTM D 2369
Pigment Content of Solvent-Type Paints	ASTM D 2371
Infrared Identification of Vehicle	
Solids From Solvent-Type Paints	ASTM D 2621
Volume Nonvolatile Matter in Clear or Pigmented Coatings	ASTM D 2697
Vehicle Solids (Ordinary Centrifuge)	FTMS 141 Method 4051
Nonvolatile Vehicle Content	FTMS 141 Method 4053

Sampling and testing performed by the Contracting Agency shall not be construed as determining or predicting the performance or compatibility of the individual coating material, or the completed coating system.

The Contractor shall furnish to the Engineer five gallons of finish coat paint in the appropriate color specified in the Special Provisions as a part of this contract. The paint container shall be marked to show the lot number, bridge number and paint name and color number.

6-07.3(2)F Preparing Paint Materials for Use

Coating materials will be rejected if:

a) The material arrives at the application site in other than the original, unopened containers.
b) The container has a break in the lid seal or a puncture.
c) The coating material has begun to polymerize, solidify, gel, or deteriorate in any manner.
d) The recommended shelf life, as stated on the manufacturer's product data sheets, has expired.
e) A skin forms on the surface of the material or on the sides of the container and the volume of the skin exceeds 2 percent of the material. If there is not more than 2 percent skin, the Contractor shall remove and discard only the skin.

Mixing

The Contractor shall thoroughly mix coating materials by mechanical means to ensure a uniform composition. Coating materials shall not be mixed by means of air stream bubbling or boxing. Coating materials shall be mixed in the original containers and mixing shall continue until all pigment or metallic powder is in suspension. Care shall be taken to ensure that the solid coating material that has settled to the bottom of the container is thoroughly dispersed. After mixing, the Contractor shall inspect the coating materials for uniformity and to ensure that no unmixed pigment or lumps are present.

Catalysts, curing agents, hardeners, initiators, or dry metallic powders which are packaged separately shall be added to the base coating material only after the base coating material is thoroughly mixed to achieve a uniform mixture with all particles wetted. The Contractor shall then add the proper volume of curing agent to the correct volume of base and mix thoroughly. The mixture shall be used within the pot life specified by the manufacturer. Unused portions shall be discarded at the end of each work day.

Thinning

The Contractor shall not add additional thinner at the application site except as approved by the Engineer. The amount and type of thinner, if allowed, shall conform to the manufacturer's specifications.

Application Site Tinting

Application site tinting will not be allowed except as approved by the Engineer.

Agitators

When recommended by the manufacturer, the Contractor shall constantly agitate coating materials during application by use of paint pots equipped with mechanical agitators.

6-07.3(2)G Painting Steel Surfaces

The coating system for all steel surfaces shall incorporate three single component moisture-cured polyurethane coats. The first component shall be the primer coat, Standard Formula A-11-99. This coat shall be used as a spot coat in areas that are cleaned down to bare metal. The second coat shall be Standard Formula B-11-99 and third coat shall be Standard Formula C-11-99. The second and third coats shall encapsulate the entire structures.

In addition to the requirements of the Specifications, coating applications shall conform to:

a) The best practices of the trade.

b) The written recommendations of the coating manufacturer.

c) All applicable portions of the SSPC-PA1.

No primer paint shall be applied to any surface until the surface has been inspected and approved by the Engineer. Any area to which primer paint has been applied without the Engineer's inspection and approval will be considered improperly cleaned. The unauthorized application shall be completely removed and the entire area recleaned to the satisfaction of the Engineer. After the area has been recleaned, inspected, and approved, the Contractor may again initiate the painting sequence.

No additional compensation or extension of time in accordance with Section 1-08.8 will be allowed for the removal of any unauthorized paint application and recleaning of the underlying surface.

Surface Condition

The surface to be covered with a coating shall be free of dust, grease, or other substance that would prevent the bond of the succeeding application. The Contractor shall protect freshly coated surfaces from contamination by abrasives, dust, or foreign materials from any other source. The Contractor shall prepare contaminated surfaces to the satisfaction of the Engineer before applying another coat.

Application Methods

The Contractor shall apply coating materials by air or airless spray, brush, roller, any combination of these methods, or as recommended by the coating material manufacturer, unless otherwise specified. All application techniques shall conform to Section 7, SSPC-PA 1.

Each coat shall be applied in a uniform layer, completely covering the preceding coat. Individual coats shall be tinted a sufficiently different shade so that each coat can be easily detected. The Contractor shall correct runs, sags, skips, or other deficiencies before application of succeeding coats. Such corrective work may require recleaning, application of additional coating, or other means as determined by the Engineer at no additional cost to the Contracting Agency.

Painters, using brushes, shall work from pails containing a maximum of two gallons of paint. This is intended to minimize the impact of any spill.

Paint shall be stored and mixed in a secure, contained location to eliminate the potential for spills into State waters, and onto the ground and highway surfaces.

Environmental Conditions

Apply coating materials only during periods when:

1. Air temperature is above 35ºF.
2. Steel surface temperature is between 35ºF and 115ºF.
3. Steel surface does not show wet drops and is not wet.
4. Relative humidity is within the manufacturer's recommended range.

Application will not be allowed if the Engineer determines that conditions are not favorable for proper application and performance of the coating.

During painting operations the area below the bridge shall be protected with a drip tarp as specified in Section 6-07.3(2)D.

If fresh coatings are damaged by the elements, the Contractor shall replace or repair the coating to the satisfaction of the Engineer at no additional cost to the Contracting Agency.

Cleaning of equipment shall not be done in State waters nor shall resultant cleaning runoff be allowed to enter State waters. No paint cans, lids, brushes, or other debris shall be allowed to enter State waters.

Solvents, paints, paint sludge, cans, buckets, rags, brushes, and other waste associated with this project shall be collected and disposed of off site.

Paint products, petroleum products or other deleterious material shall not be wasted into, or otherwise enter, State waters as a result of project activities.

Application of Coatings

After applying the spot prime coat to all areas cleaned to bare metal and before applying the intermediate coat, the Contractor shall apply a stripe coat on all edges, corners, seams, crevices, interior angles, junction of joint members, rivet or bolt heads, nuts and threads, weld lines, and any similar surface irregularities. The stripe coat shall be the same formula as the intermediate coat. The stripe coat shall be of sufficient thickness to completely hide the surface being covered and shall be followed as soon as practical by the application of intermediate coat to its specified thickness. All stripe coats shall be done by brush.

If the spot prime coat leaves unsealed cracks or crevices, these shall be sealed with single component urethane sealant meeting the requirements of Federal specification TT-S-00230C, Type II, Class A (applied per the manufacturer's recommendation) before the intermediate coat is applied.

Coating thickness measurements will be made by the Engineer after the application of each coat and before the application of the succeeding coat. In addition, the Engineer will inspect for uniform and complete coverage and appearance. One hundred percent of all thickness measurements shall be the minimum wet film thickness specified in Section 6-07.3(5). If thickness measurements or visual inspection of coverage do not meet the specified minimum, the Contractor shall make additional applications, as necessary, to achieve thickness and coverage requirements.

In areas where wet film thickness measurements are impractical, dry film thickness measurements will be made using magnetic dry film thickness gauges as specified in Section 6-07.3(5).

If a question arises about an individual coat thickness or coverage, it will be verified by the use of a Tooke gage. If the Tooke gage shows a coat thickness to be less than a minimum dry film thickness of 3.0 mils or indicates a missing intermediate coat, the total coating system will be rejected, even if the thickness of the total system equals or exceeds the total thickness specified.

If roadway or sidewalk planks lie so close to the metal that they prevent proper cleaning and painting, the Contractor shall remove or cut the planks to provide at least a 1-inch clearance. Any plank removal or cutting shall be done as approved by the Engineer. The Contractor shall replace all planks after painting. If removal breaks or damages the planks and makes them unfit for reuse, the Contractor shall replace them at no expense to the Contracting Agency.

6-07.3(3) Painting Timber Structures

6-07.3(3)A Number of Coats and Color

Unless the Plans state otherwise:

1. Rails and rail posts on timber bridges shall receive two coats (with the wheel guard painted only on its top edge and roadway side).
2. Other timber work shall receive three coats (if the Plans or Special Provisions require it to be painted).

Paint color shall be as the Plans, Special Provisions, or Engineer may require.

6-07.3(3)B Application

As it is painted, any wood surface must be thoroughly dry and free from oil and dirt. Paint shall be applied by brush, spread evenly, and worked thoroughly into all seasoning cracks, corners, and recesses. No later coat shall be applied until the full thickness of the previous coat has dried.

Final brush strokes with aluminum paint shall be made in the same direction to ensure that powder particles “leaf” evenly.

If a painted surface has been stained by creosote nearby, it shall be given one or more coats of an approved shellac before repainting.

6-07.3(3)C Painting Treated Timber

Timber treated with creosote or oil-borne, pentachlorophenol preservatives shall normally not be painted.

Timber treated with water-borne preservatives shall be clean and be reduced to no more than 18 percent moisture content before it is painted. Any visible salt crystals on the wood surface shall be washed and brushed away — with the moisture content reduced again to the specified level before painting. Stored timber awaiting painting shall be covered and stacked with spreaders to ensure air circulation.

6-07.3(4) Painting Galvanized Surfaces

All galvanized surfaces specified to be painted shall be prepared for painting in accordance with the ASTM D 2092. The method of preparation shall be as agreed upon by the paint manufacturer and the galvanizer. The Contractor shall not begin painting until receiving the Engineer’s approval of the prepared galvanized surface.

Environmental Conditions

Steel surfaces shall be:

- Greater than 35ºF and
- Less than 115ºF

or per the manufacturer's recommendations, whichever is more stringent.

The Contractor shall paint the dry surface as follows:

	Paint Formulas	Type
First Coat	MIL-P-24441	Epoxy polyamide
Second Coat	C-11-99	Moisture Cured Aliphatic Polyurethane

Each coat shall be dry before the next coat is applied. All coats applied in the shop shall be dried hard before shipment.

6-07.3(5) Paint — Film Thickness

The paint film thickness for the paint system of Section 6-07.3(1) shall be as follows. The dry film thickness of the primer coat on the faying surfaces and on the top flanges where the welded shear connectors have been attached shall not be less than 2.5 mils nor greater than 3.5 mils. On all other areas, the minimum dry film thickness for the primer coat shall be 2.5 mils. The minimum dry film thickness for the intermediate coat shall be 3.5 mils. The minimum dry film thickness for the top coat shall be 1.0 mils.

The paint film thickness for the paint system of Section 6-07.3(2) shall be as follows. The minimum wet film thickness of each coat (primer, intermediate, and finish) shall be 6.0 mils.

If the Contract calls for the use of Formula A-5-61, the dry film thickness shall be between 0.4 and 0.7 mils. (The rapid solvent release in this vinyl pretreatment makes it difficult to measure wet film thickness.)

Any other finish, no matter how applied, shall have a wet thickness of at least 6.0 mils per coat and a dry film thickness of at least 3.0 mils per coat.

If the specified number of coats do not produce a combined dry film thickness of at least the sum of the thicknesses required per coat, the Contractor shall apply another full coat of finish paint.

Film thickness — wet and dry — will be measured by suitable gauges. The dry film thickness will be determined by the use of a magnetic or magnetic flux dry film thickness gauge. The gauge shall be calibrated on the blasted steel with plastic shims the same thickness as the minimum dry film thickness. Wet measurements will be taken immediately after the paint is applied, and dry measurements after the coat is dry and hard.

6-07.3(6) Protection of Public and Private Property

The Contractor shall protect public and private property, traffic, and other parts of the bridge (deck, sidewalks, etc.) from airborne or dripping paint. The Contractor shall supply and install enough canvas or other covering to provide this protection as painting proceeds. If the covering does not adequately protect traffic, the Engineer may require the Contractor to station lookouts who shall stop the painting while vehicles or pedestrians pass.

6-07.4 Measurement

No specific unit of measurement will apply to the lump sum price for cleaning and painting existing steel structures.

6-07.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

"Cleaning and Painting - _____", lump sum.

The lump sum contract price for "Cleaning and Painting - _____" shall be full pay for all cost in connection with furnishing and placing all necessary staging and rigging, providing material, labor, tools, and equipment, collection and storage of containment waste, collection, storage, testing, and disposal of all containment waste not conforming to the definition in Section 6-07.3(2)C, performing all cleaning and preparation of surfaces to be painted and applying all coats of paint and sealant.

"Containment of Abrasives", lump sum.

The lump sum contract price for "Containment of Abrasives" shall be full payment for all costs incurred by the Contractor in complying with the requirements as specified in Section 6-07.3(2)B to design, construct, maintain, and remove containment systems for abrasive blasting operations.

"Testing and Disposal of Containment Waste", by force account as provided in Section 1-09.6.

All costs in connection with testing containment waste, transporting containment waste for disposal, and disposing of containment waste in accordance with Section 6-07.3(2)C will be paid by force account in accordance with Section 1-09.6. For the purpose of providing a common proposal for all bidders the Contracting Agency has entered an amount for the item "Testing and Disposal of Containment Waste" in the bid proposal to become part of the total bid by the Contractor.

Payment for painting new steel structures and timber structures will be in accordance with Sections 6-03.5 and 6-04.5, respectively.

6-08 WATERPROOFING

6-08.1 Description

This work shall consist of applying waterproofing materials to Portland cement concrete surfaces as required by the Plans, these Specifications, or the Engineer. The application of these waterproofing materials will not be required if a concrete admixture meeting the requirements of 9-23.8 is used.

6-08.2 Materials

Materials shall meet the requirements of the following sections:

Asphalt for Waterproofing	9-11.1
Waterproofing Fabric	9-11.2
Portland Cement Mortar	9-11.3
Waterproofing Admixture	9-23.8

6-08.3 Construction Requirements

6-08.3(1) Storage of Fabric

The fabric shall be stored in a dry, protected place. Rolls shall not be stored standing on end.

6-08.3(2) Preparation of Surface

Concrete surfaces shall be reasonably smooth and without projections or holes that might puncture the waterproofing membrane. The surfaces shall be dry, with all dust and loose material removed. The Contractor shall not apply waterproofing in wet weather or when the air temperature is below 35ºF unless the Engineer approves in writing.

6-08.3(3) Application of Waterproofing

Waterproofing asphalt shall be stirred frequently as it is heated to between 300ºF and 350ºF. Each heating kettle shall have a thermometer.

Each coat of primer or asphalt shall begin at the low point of the surface so that water will run over (not against or along) the laps.

In applying the waterproofing, the Contractor shall:

1. Apply a coat of primer and let it dry before applying the first asphalt coat.
2. Mop hot asphalt on a band about 20-inches wide across the full length of the surface.
3. Immediately roll a starter strip of half-width fabric into the asphalt, pressing it into place to rid it of all air bubbles and to conform it closely to the surface.
4. Mop hot asphalt over the starter strip and an adjacent section of surface so that the fresh asphalt forms a band slightly wider than the full width of the fabric.
5. Immediately roll a full-width strip of fabric into the fresh asphalt, pressing it into place as before.
6. Mop hot asphalt on the latest strip and on an adjacent band of the surface slightly wider than the full width of the fabric.
7. Immediately roll another strip of fabric into the asphalt, lapping the earlier strip by at least 2-inches and pressing it into place as before.
8. Repeat steps 6 and 7 until the entire surface is covered.
9. Mop the entire surface with a final coating of hot asphalt.

The three complete moppings of asphalt shall ensure that no fabric layer ever touches another fabric layer or the concrete surface. The Contractor shall examine all laps and ensure that they are thoroughly sealed down.

Each mopping shall cover completely, with a coat heavy enough to hide the fabric weave and all gray spots from the concrete. On horizontal surfaces, at least 12 gallons of asphalt shall be used for every 100 square feet of finished work. On vertical surfaces, at least 15 gallons per 100 square feet shall be used.

At the end of each day's work, all fabric that was laid shall have received its final mopping of asphalt.

Wherever the membrane ends or is punctured by drains, pipes, etc., the Contractor shall seal the area to prevent water from entering between the waterproofing and the concrete surface.

All flashing (at curbs, against girders, spandrel walls, etc.) shall be made of separate sheets that lap the main membrane by at least 12-inches. Flashing shall be sealed closely: (1) with full metal flashing, or (2) by imbedding its upper edges in a groove poured full of an acceptable joint cement.

At each expansion joint, the membrane shall not be broken but shall be folded to permit movement. At either end of the bridge, the membrane shall run well down abutments and shall allow for expansion and contraction.

6-08.3(4) Protection Course

If the Plans require, the Contractor shall place a layer of mortar at least $1^1/_2$-inches thick over the whole surface of the membrane just after it has cooled to air temperature. This layer shall be a mix of one part Portland cement to two parts sand. It shall be distributed evenly over the membrane, tamped gently into place, finished by hand to a smooth, hard surface, then covered and kept moist for one week.

6-08.4 Measurement

Measurement will be the number of square yards of the surface of the waterproofed area.

6-08.5 Payment

Payment will be made in accordance with Section 1-04.1, for the following bid item when it is included in the proposal:

"Waterproofing", per square yard.

Waterproofing of construction joints not shown in the Plans shall be at the Contractor's expense.

6-09 MODIFIED CONCRETE OVERLAYS

6-09.1 Description

This work consists of scarifying concrete bridge decks, preparing and repairing bridge deck surfaces designated and marked for further deck preparation, and placing, finishing, and curing modified concrete overlays.

6-09.2 Materials

Materials shall meet the requirements of the following Sections:

Portland Cement	9-01.2(1)
Fine Aggregate	9-03.1(2)B
Coarse Aggregate	9-03.1(4)C
Burlap Cloth	9-23.5
Admixtures	9-23.6
Fly Ash	9-23.9
Microsilica Fume	9-23.11
Water	9-25.1

Portland cement shall be either Type I or Type II. Type III portland cement will not be allowed.

Fine aggregate shall be Class 1. Coarse aggregate shall be AASHTO grading No. 7 or No. 8.

Fly ash shall be Class F only.

Microsilica admixture shall be either a dry powder or a slurry admixture. Microsilica will be accepted based on submittal to the Engineer of a Manufacturer's Certificate of Compliance conforming to Section 1-06.3. If the microsilica is a slurry admixture, the microsilica content of the slurry shall be certified as a percent by mass.

Latex admixture shall be a non-toxic, film-forming, polymeric emulsion in water to which all stabilizers have been added at the point of manufacture. The latex admixture shall be homogeneous and uniform in composition, and shall conform to the following:

Polymer Type Styrene Butadiene

Stabilizers:

Latex	Non-ionic surfactants
Portland Cement	Polydimethyl siloxane
Percent Solids	46.0 to 49.0
Weight per Gallon	8.4 pounds at 77ºF
Color	White
PH (as shipped)	9 minimum
Freeze/Thaw Stability	5 cycles (5ºF to 77ºF)
Shelf Life	2 years minimum

Latex admixture will be accepted based on submittal to the Engineer of a Manufacturer's Certificate of Compliance conforming to Section 1-06.3.

High Molecular Weight Methacrylate (HMWM) resin for crack and joint sealing shall conform to the following:

Viscosity	<25 cps (Brookfield RVT with UL adaptor, 50rpm at 77F)… California Test 434
Density	8.5 to 8.8 pounds per gallon at 77F… ASTM D 2849
Flash Point	>200F, PMCC (Pinsky-Martens CC)
Vapor Pressure	<0.04-inches Hg at 77F, ASTM D 323
Tg (DSC)	>136F, ASTM D 3418
Gel Time	60 minutes minimum

The promoter/initiator system for the methacrylate resin shall consist of a metal drier and peroxide.

Sand for abrasive finish shall be crushed sand, oven dried, and stored in moisture proof bags. The sand shall conform to the following gradation:

Sieve Size	Percent Passing	
	Minimum	Maximum
U.S. No. 10	98	100
U.S. No. 16	55	75
U.S. No. 20	30	50
U.S. No. 30	8	25
U.S. No. 50	0	5
U.S. No. 100	0	3

All percentages are by weight.

6-09.3 Construction Requirements

6-09.3(1) Equipment

6-09.3(1)A Power Driven Hand Tools

Power driven hand tools may be used for concrete scarification in areas not accessible to scarification machines, and for further deck preparation work, except for the following:

1. Jack hammers more forceful than the nominal 30-pound class.
2. Chipping hammers more forceful than the nominal 15-pound class.

The power driven hand tools shall be operated at angles less than 45 degrees as measured from the surface of the deck to the tool.

6-09.3(1)B Rotary Milling Machines

Rotary milling machines shall be capable of scarifying a minimum width of four feet per pass, have a maximum operating weight of 50,000 pounds, and conform to the requirements in Section 1-07.7.

Machines known to meet these specifications will be specified in the Special Provisions.

6-09.3(1)C Hydro-Demolition Machines

Hydro-demolition machines shall consist of filtering and pumping units operating in conjunction with a remote-controlled robotic device. Hydro-demolition machines shall scarify a minimum width of four feet per pass, using high velocity water jets to remove $^{1}/_{2}$-inch of sound concrete with the simultaneous removal of all deteriorated concrete. Hydro-demolition machines shall also clean any exposed reinforcing steel of all rust and corrosion products.

Possible sources of machines known to meet these specifications will be specified in the Special Provisions.

6-09.3(1)D Shot Blasting Machines

Shot blasting machines shall consist of a self contained mobile unit capable of scarifying a minimum width of six feet per pass, using steel abrasive to remove $^{1}/_{2}$-inch of sound concrete. The shot blasting machine shall vacuum and store all material removed from the scarified concrete surface into a self contained unit.

Possible sources of machines known to meet these specifications will be specified in the Special Provisions.

6-09.3(1)E Air Compressor

Air compressors shall be equipped with oil traps to eliminate oil from being blown onto the roadway deck during sandblasting and air cleaning.

6-09.3(1)F Vacuum Machine

Vacuum machines shall be capable of collecting all dust, concrete chips, freestanding water and other debris encountered while cleaning during deck preparation. The machines shall be equipped with collection systems that allow the machines to be operated in air pollution sensitive areas and shall be equipped to not contaminate the deck during final preparation for concrete placement.

6-09.3(1)G Water Spraying System

The water spraying system shall include a portable high-pressure sprayer with a separate water supply of potable water. The sprayer shall be readily available to all parts of the deck being overlaid and shall be able to discharge water in a fine mist to prevent accumulation of free water on the deck. Sufficient water shall be available to thoroughly soak the deck being overlaid and to keep the deck wet prior to concrete placement.

The Contractor shall certify that the water spraying system meets the following requirements:

Pressure	2,200 psi minimum
Flow Rate	4.5 gpm minimum
Fan Tip	15° to 25° Range

6-09.3(1)H Mobile Mixer for Latex Modified Concrete

Proportioning and mixing shall be accomplished in self-contained, self-propelled, continuous-mixing units conforming to the following requirements:

1. The mixer shall be equipped so that it can be grounded.
2. The mixer shall be equipped to provide positive measurement of the portland cement being introduced into the mix. An approved recording meter, visible at all times and equipped with a ticket printout, shall be used.

3. The mixer shall be equipped to provide positive control of the flow of water and latex admixture into the mixing chamber. Water flow shall be indicated by an approved flow meter with a minimum readability of one-half gallon per minute, accurate to ± 1 percent. The water system shall have a bypass valve capable of completely diverting the flow of water. Latex flow shall also be indicated by an approved flow meter with a minimum readability of two gallons per minute, accurate to ± 1 percent. The latex system shall be equipped with a bypass valve suitable for obtaining a calibrated sample of admixture.
4. The mixer shall be equipped to be calibrated to automatically proportion and blend all components of the specified mix on a continuous or intermittent basis as required by the finishing operation, and shall discharge mixed material through a conventional chute directly in front of the finishing machine.

Inspection of each mobile mixer shall be done by the Contractor in the presence of the Engineer and in accordance with the following requirements:

1. Check the manufacturer's inspection plate or mix setting chart for the serial number, the proper operating revolutions per minute (rpm), and the approximate number of counts on the cement meter to deliver 94 pounds of cement.
2. Make a general inspection of the mobile mixer to ensure cleanliness and good maintenance practices.
3. Check to see that the aggregate bins are empty and clean and that the bin vibrators work.
4. Verify that the cement aeration system operates, that the vent is open, and that the mixer is equipped with a grounding strap. Check the cement meter feeder to ensure that all fins and pockets are clean and free from accumulated cement. If the operator cannot demonstrate, through visual inspection, that the cement meter feeder is clean, all cement shall be removed from the bin and the cement meter feeder inspected. The aeration system shall be equipped with a gauge or indicator to verify that the system is operating.
5. Verify that the main belt is clean and free of any accumulated material.
6. Check the latex strainer to ensure cleanliness.

The initial calibration shall consist of the following items:

1. Cement Meter
   a. Refer to the truck manufacturer's mix setting chart to determine the specified operating rpm and the approximate number of counts required on the cement meter to deliver 94 pounds of cement.
   b. Place at least 40 bags (about 4,000 pounds) of cement in the cement bin.
   c. Ensure the mixer is resting on a level surface.
   d. Ensure the mixer is grounded.
   e. Adjust the engine throttle to obtain the specified rpm. Operate the unit, discharging cement until the belt has made one complete revolution. Stop the belt. Reset the cement meter to zero. Position a suitable container to catch the cement and discharge approximately one bag of cement. With a stopwatch, measure the time required to discharge the cement. Record the number of counts on the cement meter and determine the weight of the cement in the container. Repeat the process of discharging approximately one bag of cement until six runs have been made. Reset the cement meter to zero for each run.

Example:

Run No.	**Cement Counts**	**Weight of Cement**	**Time In Seconds**
1	66	95	31
2	68	96	31.2
3	67	95.5	31.0
4	66	95	29.8
5	67	95.25	30.5
6	66	95	30.8
TOTAL	400	571.75	184.3

Pounds of cement per count on cement meter:

$$\frac{\text{Weight of Cement}}{\text{No. of Counts}} = \frac{571.75}{400} = 1.43 \text{ LB./Count}$$

Counts per bag (94 pounds):

$$\frac{94}{1.43} = 65.7 \text{ Counts Bag}$$

Pounds of cement discharged per second:

$$\frac{\text{Weight of Cement}}{\text{Time in Seconds}} = \frac{571.75}{184.3} = 3.10 \text{ LB./SEC.}$$

Required time to discharge one bag:

$$\text{Time} = \frac{94}{3.10} = 30.32 \text{ SEC./Bag}$$

2. Latex Throttling Valve
   a. Check to be sure that the latex strainer is unobstructed.
   b. The latex throttling valve shall be adjusted to deliver 3.5 gallons of latex (29.4 pounds) for each bag of cement. From the above calculation 30.32 seconds are required to deliver one bag of cement.
   c. With the unit operating at the specified rpm, discharge latex into a container for 30.3 seconds and determine the weight of latex. Continue adjusting the valve until 29.4 to 29.5 pounds of latex is discharged in 30.3 seconds. Verify the accuracy of this valve setting three times.
3. Water Flow Meter
   a. Set the water flow meter by adjusting it to flow at one-half gallon per minute.

b. Collect and weigh the water discharged during a one minute interval with the equipment operating at the specified rpm. Divide the weight of water by 8.34 to determine the number of gallons.

c. Repeat Items a. and b., above, with the flow meter adjusted to one and one-half gallons per minute.

4. Aggregate Bin Gates

a. Set the gate openings to provide the amount of aggregate required to produce concrete having the specified proportions.

b. Discharge a representative sample of the aggregates through the gates and separate on the U.S. No. 4 sieve. Aggregates shall meet the requirements for proportions in accordance with Section 6-09.3(3)E.

c. Adjust the gate openings if necessary to provide the proper ratio of fine aggregate to total aggregate.

5. Production of Trial Mix

Each mobile mixer shall be operated to produce at least $^{1}/_{2}$ cubic yard of concrete, which shall be in compliance with these specifications, prior to acceptance of the mobile mixer for job use. The Engineer will perform yield, slump, and air tests on the concrete produced by each mixer. Calibration of each mobile mixer shall be done by the Contractor in the presence of the Engineer. A complete calibration is required on each mixer on each concrete placement unless, after the initial calibration, the personnel having the responsibility of mixer calibration on subsequent concrete placement were present during the initial calibration of the mixer and during the concrete placement operations and are able to verify the dial settings of the initial calibration and concrete placement.

If these criteria are met, a complete calibration need not be repeated provided that a single trial run verifies the previous settings of the cement meter, latex throttling valve, water flow meter, and aggregate gradations, and that the mixer has not left the project and the Engineer is satisfied that a complete calibration is not needed.

6-09.3(1)I Ready Mix Trucks for Fly Ash Modified and Microsilica Modified Concrete

Ready mix trucks shall conform to Section 6-02.3(4)A.

6-09.3(1)J Finishing Machine

The finishing machine shall meet the requirements of Section 6-02.3(10) and the following requirements:

The finishing machine shall be equipped with a rotating cylindrical double drum screed not exceeding 60-inches in length preceded by a vibrating pan. The vibrating pan shall be constructed of metal and be of sufficient length and width to properly consolidate the mixture. The vibrating frequency of the vibrating pan shall be variable with positive control between 3,000 and 6,000 rpm. A machine with a vibrating pan as an integral part may be proposed and will be considered for approval by the Engineer. Other finishing machines will be allowed subject to approval of the Engineer.

6-09.3(2) Submittals

The Contractor shall submit the following items to the Engineer for approval in accordance with Section 6-01.9:

1. The type of machine (rotary milling, hydro-demolition, or shot blasting) selected by the Contractor for use in this project to scarify concrete surfaces.
2. The axle loads and axle spacing of the rotary milling machine (if used).
3. The Runoff Water Disposal Plan (if a hydro-demolition machine is used). The Runoff Water Disposal Plan shall describe all provisions for the containment, collection, filtering, and disposal of all runoff water and associated contaminants generated by the hydro-demolition process.
4. The method and materials used to contain, collect, and dispose of all concrete debris generated by the scarifying process, including provisions for protecting adjacent traffic from flying debris.
5. The mix design for concrete Class M, and either fly ash modified concrete, microsilica modified concrete, or latex modified concrete, as selected by the Contractor for use in this project in accordance with Section 6-09.3(3).
6. Samples of the latex admixture and the portland cement for testing and compatibility (if latex modified concrete is used).
7. Details of the screed rail support system, including details of anchoring the rails and providing rail continuity.

The Contractor shall not begin scarifying operations until receiving the Engineer's approval of Items 1 through 4 as applicable for the Contractor's scarifying method. The Contractor shall not begin placing modified concrete overlay until receiving the Engineer's approval of Items 5 through 7 as applicable for the Contractor's selected type of modified concrete.

6-09.3(3) Concrete Overlay Mixes

6-09.3(3)A General

For fly ash, microsilica, and latex modified concrete, the Contractor shall adjust the slump to accommodate the gradient of the bridge deck, subject to the maximum slump specified.

For fly ash and microsilica modified concrete, the maximum water/cement ratio shall be calculated using all of the available mix water, including the free water in both the coarse and fine aggregate, and in the microsilica slurry if a slurry is used.

For fly ash and microsilica modified concrete, all water reducing and air entraining admixtures, and superplasticizers, shall be used in accordance with the fly ash supplier's and microsilica admixture supplier's recommendations, respectively, and as approved by the Engineer.

6-09.3(3)B Concrete Class M

Concrete Class M for further deck preparation patching concrete shall be proportioned in accordance with the following mix design:

Portland Cement	705 pounds
Fine Aggregate	1,280 pounds
Coarse Aggregate	1,650 pounds
Water/Cement Ratio	0.37 maximum
Air (± 1$^1/_2$ percent)	6 percent
Slump (± 1-inch)	5-inches

The use of a water-reducing admixture conforming to AASHTO M 194 Type A will be required to produce patching concrete with the desired slump, and shall be used in accordance with the admixture manufacturer's recommendations. Air entraining admixtures shall conform to AASHTO M 154 and shall be used in accordance with the admixture manufacturer's recommendations. The use of accelerating admixtures or other types of admixtures is not allowed.

6-09.3(3)C Fly Ash Modified Concrete

Fly ash modified concrete shall be a workable mix, uniform in composition and consistency. Mix proportions per cubic yard shall be as follows:

Portland Cement	611 pounds
Fly Ash	275 pounds
Fine Aggregate	38 percent of total aggregate
Coarse Aggregate	62 percent of total aggregate
Water/Cement Ratio	0.30 maximum
Air (± 1$^1/_2$ percent)	6 percent
Slump	7-inches maximum

6-09.3(3)D Microsilica Modified Concrete

Microsilica modified concrete shall be a workable mix, uniform in composition and consistency. Mix proportions per cubic yard shall be as follows:

Portland Cement	658 pounds
Microsilica Fume	52 pounds
Fine Aggregate	1,515 pounds
Coarse Aggregate	1,515 pounds
Water/Cement Ratio	0.33 maximum
Air (± 1$^1/_2$ percent)	6 percent
Slump	7-inches maximum

6-09.3(3)E Latex Modified Concrete

Latex modified concrete shall be a workable mix, uniform in composition and consistency. Mix proportions per cubic yard shall be as follows:

Portland Cement	1.00 parts by weight
Fine Aggregate	2.40 to 2.75 parts by weight
Coarse Aggregate	1.75 to 2.00 parts by weight
Latex Admixture	3.50 gallons per bag of cement
Water/Cement Ratio	0.33 maximum
Air Content of Plastic Mix	6 percent maximum
Slump	7inches maximum

The aggregates shall be proportioned such that the amount of aggregate passing the U.S. No. 4 sieve is 65 ± 5 percent of the total aggregate (fine plus coarse). All calculations shall be based on dry weights.

The moisture content of the fine aggregate and coarse aggregate shall be no more than 3.0 and 1.0 percent, respectively, above the saturated surface dry condition.

The water limit for calculating the water/cement ratio shall include the added water, the free water in the aggregates, and 52 percent of the latex admixture.

6-09.3(4) Storing and Handling

6-09.3(4)A Aggregate

Aggregates shall be stored and handled in a manner to prevent variations of more than 1.0 percent in moisture content of the stockpile.

For latex modified concrete, the moisture content of the aggregate at the time of proportioning shall be as specified in Section 6-09.3(3)E.

6-09.3(4)B Latex Admixture

The admixture shall be kept in suitable containers that will protect it from freezing and from exposure to temperatures in excess of 85ºF. Containers of the admixture shall not be stored in direct sunlight for periods in excess of ten days. When stored in direct sunlight the top and sides of the containers shall be covered with insulating blanket material.

Storage of the admixture may extend over a period greater than ten days as long as the conditions specified above are maintained and the latex admixture is agitated or stirred once every ten days. Stirring or agitation of the admixture shall be done mechanically in accordance with the manufacturer's recommendation and as approved by the Engineer. If the ambient temperature is higher than 85ºF at any time during the storage period, the admixture shall be covered by insulated blankets or other means that will maintain the admixture temperature below 85ºF.

The admixture shall be strained through a Number 10 strainer at the time it is introduced into the mixing tank from the storage containers.

6-09.3(4)C High Molecular Weight Methacrylate Resin (HMWM)

The HMWM resin shall be stored in a cool dry place and protected from freezing and exposure to temperature in excess of 100ºF. The promoter and initiator, if supplied separate from the resin, shall not contact each other directly. Containers of promoters and initiators shall not be stored together in a manner that will allow leakage or spillage from one to contact the containers or material of the other.

6-09.3(5) Scarifying Concrete Surface

6-09.3(5)A General

The Contractor shall not begin scarifying a concrete bridge deck surface unless completion of the scarification and concrete overlay can be accomplished within the current construction season.

The Contractor shall not begin scarifying a concrete bridge deck surface until receiving the Engineer's written approval of the machine to be used for scarifying

The Contractor shall protect adjacent traffic from flying debris generated by the scarification process in accordance with Item 4 of Section 6-09.3(2) and as approved by the Engineer.

The Contractor shall collect, contain, and dispose of all concrete debris generated by the scarification process in accordance with Item 4 of Section 6-09.3(2) and as approved by the Engineer.

All areas of the deck that are inaccessible to the selected scarifying machine shall be scarified to remove the concrete surface matrix to a maximum depth of $^1/_2$-inch by a method approved by the Engineer. If these areas are hand-chipped then the equipment shall meet the requirements as specified in Section 6-09.3(1)A.

6-09.3(5)B Testing of Hydro-Demolition and Shot Blasting Machines

A trial area shall be designated by the Engineer to demonstrate that the equipment and methods of operation are capable of producing results satisfactory to the Engineer. The trial area shall consist of two patches each of approximately 30 square feet, one area in sound concrete and one area of deteriorated concrete as determined by the Engineer.

In the "sound" area of concrete, the equipment shall be programmed to remove $^1/_2$-inch of concrete.

Following the test over sound concrete, the equipment shall be located over the deteriorated concrete and using the same parameters for the sound concrete removal, remove all deteriorated concrete. The Engineer will grant approval of the equipment based on successful results from the trial area test.

6-09.3(5)C Hydro-Demolishing

Once the operating parameters of the Hydro-Demolition machine are defined by programming and calibration as specified in Section 6-09.3(5)B, they shall not be changed as the machine progresses across the bridge deck, in order to prevent the unnecessary removal of sound concrete below the required minimum removal depth. The Contractor shall maintain a minimum production rate of 250 square feet per hour during the deck scarifying process.

All water used in the Hydro-Demolition process shall be potable. Stream or lake water will not be permitted.

All bridge drains and other outlets within 100-feet of the Hydro-Demolition machine shall be temporarily plugged during the Hydro-Demolition operation. When scarifying a bridge deck passing over traffic lanes, the Contractor shall protect the traffic below by restricting and containing scarifying operations, and implementing traffic control measures, as approved by the Engineer.

The Contractor shall provide for the collection, filtering and disposal of all runoff water generated by the Hydro-Demolition process, in accordance with the Runoff Water Disposal Plan as approved by the Engineer in accordance with Item 3 of Section 6-09.3(2). The Contractor shall comply with applicable regulations concerning such water disposal.

6-09.3(5)D Shot Blasting

Once the operating parameters of the Shot Blasting machine are defined by programming and calibration, as specified in Section; 6-09.3(5)B, they shall not be changed as the machine progresses across the bridge deck, in order to prevent the unnecessary removal of sound concrete below the required minimum removal depth. The Contractor shall maintain a minimum production rate of 250 square feet per hour during the deck scarifying process.

6-09.3(5)E Rotomilling

The entire concrete surface of the bridge deck shall be scarified to remove the surface matrix to a maximum 1/2-inch depth of the concrete. The operating parameters of the rotary milling machine shall be monitored in order to prevent the unnecessary removal of sound concrete below the 1/2-inch maximum removal depth.

6-09.3(5)F Repair of Steel Reinforcing Bars Damaged by Scarifying Operations

All reinforcing steel damaged due to the Contractor's operations shall be repaired by the Contractor. For bridge decks not constructed under the same contract as the concrete overlay, damage to existing reinforcing steel shall be repaired and paid for in accordance with Section 1-09.6 if the existing concrete cover is 1/2-inch or less. All other reinforcing steel damaged due to the Contractor's operations shall be repaired by the Contractor at no additional expense to the Contracting Agency.

The repair shall be as follows or as directed by the Engineer:

1. Damage to epoxy coating, when present on existing steel reinforcing bars, shall be repaired in accordance with Section 6-02.3(24)H.
2. Damage to bars resulting in a section loss of 20 percent or more of the bar area shall be repaired by chipping out the adjacent concrete and splicing a new bar of the same size. Concrete shall be removed to provide a 3/4-inch minimum clearance around the bars. The splice bars shall extend a minimum of 40 bar diameters beyond each end of the damage.
3. Any bars partially or completely removed from the deck shall have the damaged portions removed and spliced with new bars as outlined in Item 2 above.

6-09.3(5)G Cleanup Following Scarification

After scarifying is completed, the lane or strip being overlaid shall be thoroughly cleaned of all dust, freestanding water and loose particles. Cleaning may be accomplished by using compressed air, water blasting, with a minimum pressure of 5,000 psi, or vacuum machines. Vacuum cleaning shall be used when required by applicable air pollution ordinances.

6-09.3(6) Further Deck Preparation

Once the lane or strip being overlaid has been cleaned of debris from scarifying, the Contractor, under the direction of the Engineer, shall perform an inspection of the completed work and shall mark those areas of the existing bridge deck that require further deck preparation by the Contractor. Further deck preparation will be required when any one of the following conditions is present:

1. Unsound concrete.
2. Lack of bond between existing concrete and reinforcing steel.
3. Exposure of reinforcing steel to a depth of one-half of the periphery of a bar for a distance of 12-inches or more along the bar.
4. Existing non-concrete patches as marked by the Engineer.

If the concrete overlay is placed on a bridge deck as part of the same contract as the bridge deck construction, then all work associated with the further deck preparation shall be performed at no additional expense to the Contracting Agency.

6-09.3(6)A Equipment for Further Deck Preparation

Further deck preparation shall be performed using either hand operated tools conforming to Section 6-09.3(1)A, or hydro-demolishing machines conforming to Section 6-09.3(1)C.

6-09.3(6)B Deck Repair Preparation

All concrete in the repair area shall be removed by chipping, hydro demolishing, or other approved mechanical means to a depth necessary to remove all loose and unsound concrete. If unsound concrete exists around the steel reinforcing bars, or if the bond between concrete and steel is broken, concrete must be removed to provide a 3/4-inch minimum clearance around the steel reinforcing bars.

Care shall be taken in removing the deteriorated concrete to not damage any of the existing deck or steel reinforcing bars that are to remain in place. All removal shall be accomplished by making neat vertical cuts and maintaining square edges at the boundaries of the repair area. Cuts made by using sawing or hydro demolishing machines shall be made after sufficient concrete removal has been accomplished to establish the limits of the removal area. In no case shall the depth of the vertical cut exceed 3/4-inch or to the top of the top steel reinforcing bars, whichever is less.

The exposed steel reinforcing bars and concrete in the repair area shall be sandblasted or hydro-blasted and blown clean just prior to placing concrete. Bridge deck areas outside the repair area or steel reinforcing bar inside or outside the repair area damaged by the Contractor's operations, shall be repaired by the Contractor at no additional expense to the Contracting Agency, and to the satisfaction of the Engineer.

All steel reinforcing bars damaged due to the Contractor's operations shall be repaired in accordance with Section 6-09.3(5)F.

6-09.3(6)C Placing Deck Repair Concrete

Patching concrete for modified concrete overlays shall be either modified concrete or concrete Class M. For small deck repair, and as determined by the Engineer, the Contractor may use the same modified concrete as that used in the overlay.

Before placing any patching concrete, the Contractor shall flush the existing concrete in the repair area with water and make sure that the existing concrete is well saturated. The Contractor shall remove any freestanding water prior to placing the patching concrete. The Contractor shall place the patching concrete other than latex modified concrete onto the existing concrete while it is wet.

If latex, fly ash, or microsilica modified concrete is used as the patching concrete, a thin slurry bond grout shall be scrubbed into the existing concrete surface. The bond grout shall match the overlay type being used as specified in Section 6-09.3(11).

If the Contractor elects to use as a patching material the same modified concrete as that used in the overlay, then the repair areas shall be filled flush with the deck surface sufficiently in advance of the overlay placement so that the material will not roll back under the screeds but shall not be placed more than one hour in advance of the overlay placement.

Areas patched with modified concrete or concrete Class M shall be wet cured for 24 hours in accordance with Section 6-09.3(13). During the curing period, all vehicular and foot traffic shall be prohibited on the repaired area.

6-09.3(7) Surface Preparation For Concrete Overlay

Following the completion of any required further deck preparation the entire lane or strip being overlaid shall be cleaned.

If either a rotary milling machine or a shot blasting machine is used for concrete scarification, then the concrete deck shall be sandblasted or shot blasted, using equipment approved by the Engineer, until sound concrete is exposed. Care shall be taken to ensure that all exposed reinforcing steel and the surrounding concrete is completely blasted. Bridge grate inlets, expansion dams and barriers above the surface to be blasted shall be protected from the blasting.

If a hydro-demolition machine is used for concrete scarification, then the concrete deck shall be cleaned by an approved method of water blasting with 7,000 psi minimum pressure, until sound concrete is exposed.

The final surface of the deck shall be free from oil and grease, rust and other foreign material that may reduce the bond of the new concrete to the old. These materials shall be removed by detergent- cleaning or other method as approved by the Engineer followed by sandblasting.

After all scarifying, chipping, sandblasting and cleaning is completed, the entire lane or strip being overlaid shall be cleaned in final preparation for placing concrete using either compressed air or vacuum machines. Vacuum machines shall be used when warranted by applicable air pollution ordinances.

Scarifying with either rotary milling machines or shot blasting machines, hand tool chipping, sandblasting and cleaning in areas adjacent to a lane or strip being cleaned in final preparation for placing concrete shall be discontinued when final preparation is begun. Scarifying and hand tool chipping shall remain suspended until the concrete has been placed and the requirement for curing time has been satisfied. Sandblasting and cleaning shall remain suspended for the first 24 hours of curing time after the completion of concrete placing.

If the hydro demolishing scarification process is used, scarification may proceed during the final cleaning and overlay placement phases of the work on adjacent portions of the structure so long as the hydro demolisher operations are confined to areas which are a minimum of 100-feet away from the defined limits of the final cleaning or overlay placement in progress. If the hydro demolisher impedes or interferes in any way with the final cleaning or overlay placement as determined by the Engineer, the hydro demolishing work shall be terminated immediately and the hydro demolishing equipment removed sufficiently away from the area being prepared or overlaid to eliminate the conflict. If the grade is such that water and contaminates from the hydro demolishing operation will flow into the area being prepared or overlaid, the hydro demolishing operation shall be terminated and shall remain suspended for the first 24 hours of curing time after the completion of concrete placement.

If, after final cleaning, the lane or strip being overlaid becomes wet, the Contractor shall flush the surface with high-pressure water, prior to placement of the overlay. All freestanding water shall be removed prior to concrete placement. Concrete placement shall begin within 24 hours of the completion of deck preparation for the portion of the deck to be overlaid. If concrete placement has not begun within 24 hours, the lane or strip being overlaid shall be cleaned by a light sand blasting followed by washing with the high-pressure water spray or by cleaning with the high-pressure spray as approved by the Engineer.

Traffic other than required construction equipment will not be permitted on any portion of the lane or strip being overlaid that has undergone final preparation for placing concrete unless approved by the Engineer. To prevent contamination, all equipment allowed on the deck after final cleaning shall be equipped with drip guards.

6-09.3(8) Quality Assurance

6-09.3(8)A Quality Assurance for Microsilica Modified and Fly Ash Modified Concrete Overlays

The Engineer will perform slump, temperature, and entrained air tests for acceptance after the Contractor indicates that the concrete is ready for placement. Concrete from the first truckload shall not be placed until tests for acceptance have been completed by the Engineer and the results indicate that the concrete is within acceptable limits. Sampling and testing will continue for each load until two successive loads meet all applicable acceptance test requirements. Except for the first load of concrete, up to $^1/_2$ cubic yard may be placed prior to testing for acceptance. After two successive tests indicate that the concrete is within specified limits, the sampling and testing frequency may decrease to one for every three truckloads. Loads to be sampled will be selected in accordance with the random selection process outlined in FOP for WAQTC TM2.

When the results of any subsequent acceptance test indicates that the concrete does not conform to the specified limits, the sampling and testing frequency will be resumed for each truckload. Whenever two successive subsequent tests indicate that the concrete is within the specified limits, the random sampling and testing frequency of one for every three truck loads may resume.

The test for determining the slump of the concrete will be conducted in accordance with the WSDOT FOP for AASHTO T 119 and the test for determining the percentage of entrained air will be conducted in accordance with the WAQTC FOP for AASHTO T 152.

The Engineer will test for slump and/or air any load of concrete the Engineer deems necessary.

6-09.3(8)B Quality Assurance for Latex Modified Concrete Overlays

The Engineer will perform operational control testing as the concrete is being placed. The Contractor shall provide the Engineer with a 1/4-cubic yard container and assistance in obtaining and handling samples. The 1/4-cubic yard container shall have a 9-inch minimum depth and shall be placed on a level surface. A minimum of one test per mobile mixer per shift will be conducted. The test will be conducted after eight minutes of mixer operation.

The Engineer will perform slump and air tests as the concrete is being placed. The minimum number of tests will be one slump test and one air test per mobile mixer, beginning with the first charge and every other charge thereafter. The sample will be taken after the first two minutes of continuous mixer operation. The concrete will be sampled as follows:

1. While concrete is being deposited onto the bridge deck, the stream will be diverted into a wheelbarrow or other suitable container. Approximately 1 cubic foot of concrete will be sufficient to conduct one slump test and one air test.
2. Take the sample to the test site. The test site should be located away from the mobile mixer and off the end of the bridge if practical.
3. Allow the sample to stand undisturbed. The fresh concrete sample must be protected from sunlight and wind until the conclusion of the testing. Total time from discharge to time of start of slump testing will not exceed six and one half minutes.

The test for determining the slump of the concrete will be conducted in accordance with WSDOT FOP for AASHTO T 119 and the test for determining the percentage of entrained air will be conducted in accordance with WAQTC FOP for AASHTO T 152.

During the initial proportioning, mixing, placing, and finishing operations, the Engineer may require the presence of a technical representative from the latex admixture manufacturer. The technical representative shall be capable of performing, demonstrating, inspecting, and testing all of the functions required for placement of the latex modified concrete as specified in Section 6-09.3(11) and as approved by the Engineer. This technical representative shall aid in the proper installation of the latex modified concrete. Recommendations made by the technical representative on or off the jobsite, and approved by the Engineer, shall be adhered to by the Contractor at no additional expense to the Contracting Agency. The Engineer will advise the Contractor in writing a minimum of five working days before such services are required.

6-09.3(9) Mixing Concrete For Concrete Overlay

6-09.3(9)A Mixing Microsilica Modified or Fly Ash Modified Concrete

Mixing of concrete shall be in accordance with Section 6-02, with the following exceptions:

1. The mixing shall be done at a batch plant.
2. The volume of concrete transported by truck shall not exceed six cubic yards per truck.

6-09.3(9)B Mixing Latex Modified Concrete

The equipment used for mixing the concrete shall be operated with strict adherence to the procedures set forth by its manufacturer.

A minimum of two mixers will be required at the overlay site for each concrete placement when the total volume of concrete to be placed during the concrete placement exceeds the material storage capacity of a single mixer. Additional mixers may be required if conditions require that material be stockpiled away from the jobsite. The Contractor shall have sufficient mixers on hand to ensure a consistent and continuous delivery and placement of concrete throughout the concrete placement.

Charging the mobile mixer shall be done in the presence of the Engineer. Mixing capabilities shall be such that the finishing operation can proceed at a steady pace.

6-09.3(10) Overlay Profile and Screed Rails

The overlay shall have a thickness of 1 1/2-inches or as specified by the Engineer. The thickness shall be verified prior to the placement of concrete by attaching a filler block, having a thickness of 1/4-inch less than the overlay thickness, to the bottom of the screed. The filler block shall pass freely over the surface to be overlaid. With the screed guides in place, the finishing machine shall be passed over the entire surface to be overlaid and the final screed rail adjustments shall be made.

If the overlay thickness does not verify, the profile of the new concrete surface shall be adjusted as approved by the Engineer.

After the overlay thickness has been verified, changes in the finishing machine elevation controls will not be allowed.

Rails upon which the finishing machine travels shall be placed outside of the area to be overlaid, in accordance with Item 7 of Section 6-09.3(2) and as approved by the Engineer. Interlocking rail sections or other approved methods of providing rail continuity are required.

Hold-down devices shot into the concrete are not permitted unless the concrete is to be subsequently overlaid. Hold-down devices of other types leaving holes in the exposed area will be allowed provided the holes are subsequently filled with a sand/cement grout (sand and portland cement in equal proportions by volume). Hold-down devices shall not penetrate the existing deck by more than 3/4-inch.

Rails may be removed at any time after the concrete has taken an initial set. Adequate precautions shall be taken during the removal of the finishing machine and rails to protect the edges of the new surfaces.

The Contractor shall be responsible for setting screed control to obtain the nominal overlay thickness specified as well as the finished surface smoothness requirements.

6-09.3(11) Placing Concrete Overlay

Prior to concrete placement, the Contractor shall review the equipment, procedures, personnel, and previous results with the Engineer. Inspection procedures shall also be reviewed to ensure coordination.

Concrete placement shall be made in accordance with Section 6-02 and the following requirements:

1. After the lane or strip to be overlaid has been prepared and immediately before placing the concrete, it shall be thoroughly soaked and kept continuously wet with water for a minimum period of six hours prior to placement of the concrete. All freestanding water shall be removed prior to concrete placement. During concrete placement, the lane or strip shall be kept moist.

   The concrete shall then be promptly and continuously delivered and deposited on the placement side of the finishing machine.

   If latex modified concrete is used, the concrete shall be thoroughly brushed into the surface and then brought up to final grade. If either microsilica modified concrete or fly ash modified concrete are used, a slurry of the concrete, excluding aggregate, shall be thoroughly brushed into the surface prior to the overlay placement.

Care shall be exercised to ensure that the surface receives a thorough, even coating and that the rate of progress is limited so that the brushed concrete does not become dry before it is covered with additional concrete as required for the final grade. All aggregate which is segregated from the mix during the brushing operation shall be removed from the deck and disposed of by the Contractor.

If either microsilica modified concrete or fly ash modified concrete are used, the Contractor shall ensure that a sufficient number of trucks are used for concrete delivery to obtain a consistent and continuous delivery and placement of concrete throughout the concrete placement operation.

When concrete is to be placed against the concrete in a previously placed transverse joint, lane, or strip, the previously placed concrete shall be sawed back six inches to straight and vertical edges and shall be sandblasted or water blasted before new concrete is placed. The Engineer may decrease the six inch saw back requirement to two inches minimum, if a bulkhead was used during previous concrete placement and the concrete was hand vibrated along the bulkhead.

2. Concrete placement shall not begin if rain is expected. Adequate precautions shall be taken to protect freshly placed concrete in the event that rain begins during placement. Concrete that is damaged by rain shall be removed and replaced by the Contractor at no additional expense to the Contracting Agency, and to the satisfaction of the Engineer.
3. Concrete shall not be placed when the temperature of the concrete surface is less than 45ºF or greater than 75ºF, when the combination of air temperature, relative humidity, fresh concrete temperature, and wind velocity at the construction site produces an evaporation rate of 0.15 pound per square foot of surface per hour as determined from Table 6-02.3(6)-1, or when winds are in excess of 10 mph. If the Contractor elects to work at night to meet these criteria, adequate lighting shall be provided at no additional expense to the Contracting Agency, and as approved by the Engineer.
4. If concrete placement is stopped for a period of one-half hour or more, the Contractor shall install a bulkhead transverse to the direction of placement at a position where the overlay can be finished full width up to the bulkhead. The bulkhead shall be full depth of the overlay and shall be installed to grade. The concrete shall be finished and cured in accordance with these specifications.

   Further placement is permitted only after a period of 12 hours unless a gap is left in the lane or strip. The gap shall be of sufficient width for the finishing machine to clear the transverse bulkhead installed where concrete placement was stopped. The previously placed concrete shall be sawed back from the bulkhead, to a point designated by the Engineer, to straight and vertical edges and shall be sandblasted or water blasted before new concrete is placed.
5. Concrete shall not be placed against the edge of an adjacent lane or strip that is less than 36 hours old.

6-09.3(12) Finishing Concrete Overlay

Finishing shall be accomplished in accordance with the applicable portions of Section 6-02.3(10) and as follows. Concrete shall be placed and struck-off approximately $^1/_2$-inch above final grade and then consolidated and finished to final grade with a single pass (the Engineer may require additional passes) of the finishing machine. Hand finishing may be necessary to close up or seal off the surface. The final product shall be a dense uniform surface.

Latex shall not be sprayed on a freshly placed latex modified concrete surface; however, a light fog spray of water is permitted if required for finishing, as determined by the Engineer.

As the finishing machine progresses along the placed concrete, the surface shall be given a final finish by texturing with a comb perpendicular to the centerline of the bridge. The texture shall be applied immediately behind the finishing machine. The comb shall consist of a single row of metal tines capable of producing $^1/_8$-inch wide striations approximately 0.015-foot in depth at approximately $^1/_2$-inch spacing. The combs may be operated manually or mechanically, either singly or in gangs (several combs placed end to end). This operation shall be done in a manner that will minimize the displacement of the aggregate particles. The texture shall not extend into areas within 2-feet of the curb line. The non-textured concrete within 2-feet of the curb line shall be hand finished with a steel or magnesium trowel.

Screed rails and construction dams shall be separated from the newly placed concrete by passing a pointing trowel along the inside surfaces of the rails or dams. Care shall be exercised to ensure that this trowel cut is made for the entire depth and length of rails or dams after the concrete has stiffened sufficiently that it does not flow back.

After the burlap cover has been removed and the concrete surface has dried, but before opening to traffic, all joints and visible cracks shall be filled and sealed with a high molecular weight methacrylate resin (HMWM). Cracks $^1/_{16}$-inch and greater in width shall receive two applications of HMWM. Immediately following the application of HMWM, the wetted surface shall be coated with sand for abrasive finish.

6-09.3(13) Curing Concrete Overlay

As the texturing portion of the finishing operation progresses, the concrete shall be immediately covered with a single layer of clean, new or used, wet burlap. The burlap shall have a maximum width of six feet. The Engineer will determine the suitability of the burlap for reuse, based on the cleanliness and absorption ability of the burlap. Care shall be exercised to ensure that the burlap is well drained and laid flat with no wrinkles on the deck surface. Adjacent strips of burlap shall have a minimum overlap of six inches.

Once in place the burlap shall be lightly fog sprayed with water. A separate layer of white, reflective type polyethylene sheeting shall immediately be placed over the wet burlap. The concrete shall then be wet cured by keeping the burlap wet for a minimum of 42 hours after which the polyethylene sheeting and burlap may be removed.

Traffic shall not be permitted on the finished concrete until the specified curing time is satisfied and until the concrete has reached a minimum compressive strength of 3,000 psi as verified by rebound number determined in accordance with ASTM C 805.

6-09.3(14) Checking for Bond

After the requirements for curing have been met, the entire overlaid surface shall be sounded by the Contractor, in a manner approved by and in the presence of the Engineer, to ensure total bond of the concrete to the bridge deck. Concrete in unbonded areas shall be removed and replaced by the Contractor with the same modified concrete as used in the overlay at no additional expense to the Contracting Agency. All cracks, except those that are significant enough to require removal, shall be thoroughly filled and sealed as specified in Section 6-09.3(12).

After the curing requirements have been met, the Contractor may use compressed air to accelerate drying of the deck surface for crack identification and sealing.

6-09.4 Measurement

Scarifying concrete surface will be measured by the square yard of surface actually scarified.

Modified concrete overlay will be measured by the cubic foot of material placed. For latex modified concrete overlay, the volume will be determined by the theoretical yield of the design mix and documented by the counts of the cement meter less waste. For both microsilica modified concrete overlay and fly ash modified concrete overlay, the volume will be determined from the concrete supplier's Certificate of Compliance for each batch delivered less waste. Waste is defined as the following:

1. Material not placed.
2. Material placed in excess of six inches outside a longitudinal joint or transverse joint.

Finishing and curing modified concrete overlay will be measured by the square yard of overlay surface actually finished and cured.

When further deck preparation is measured by volume, it will be measured by the cubic foot of material placed. When latex modified concrete overlay is used as the repair material, the volume will be determined by the theoretical yield of the design mix and documented by the counts of the cement meter less waste. When either microsilica modified concrete overlay, fly ash modified concrete overlay, or concrete Class M are used as the repair material, the volume will be determined from the concrete supplier's Certificate of Compliance for each batch delivered less waste.

6-09.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the bid proposal:

"Scarifying Conc. Surface", per square yard.

The unit contract price per square yard for "Scarifying Conc. Surface" shall be full pay for performing the work as specified, including testing and calibration of the machines and tools used, containment, collection, and disposal of all water and abrasives used and debris created by the scarifying operation, measures taken to protect adjacent traffic from flying debris, and final cleanup following the scarifying operation.

"Modified Conc. Overlay", per cubic foot.

The unit contract price per cubic foot for "Modified Conc. Overlay" shall be full pay for furnishing the modified concrete overlay.

"Finishing and Curing Modified Conc. Overlay", per square yard.

The unit contract price per square yard for "Finishing and Curing Modified Conc. Overlay" shall be full pay for performing the work as specified, including placing, finishing, and curing the modified concrete overlay, checking for bond, and sealing all cracks.

"Further Deck Preparation", per cubic foot.

When "Further Deck Preparation" is measured by volume, the unit contract price per cubic foot for "Further Deck Preparation" shall be full pay for performing the work as specified, including removing and disposing of the concrete within the repair area, and furnishing, placing, finishing, and curing the repair concrete.

"Further Deck Preparation", force account.

When "Further Deck Preparation" is not measured by volume, payment for the work required will be by force account in accordance with Section 1-09.6. For the purpose of providing a common proposal for all bidders, the Contracting Agency has entered an amount for the item "Further Deck Preparation" in the bid proposal to become a part of the total bid by the Contractor.

6-10 CONCRETE BARRIER

6-10.1 Description

This Section applies to building precast or cast-in-place cement concrete barriers as required by the Plans, these Specifications, or the Engineer.

This work may also include the removal, storage and resetting of permanent barrier at the locations shown in the Plans or as approved by the Engineer.

6-10.2 Materials

Materials shall meet the requirements of the following sections:

Portland Cement	9-01
Aggregates	9-03
Premolded Joint Fillers	9-04.1
Reinforcing Steel	9-07

Wire rope shall be Class 6 x 19, made of improved plow steel that has been galvanized and preformed. Galvanizing shall meet ASTM A 603. The wire rope shall have right regular lay and a fiber core. It shall be 5/8-inch in diameter and have a minimum breaking strength of 15 tons.

All hardware (connecting pins, drift pins, nuts, washers, etc.) shall be galvanized in keeping with AASHTO M 232.

Connecting pins, drift pins and steel pins for type 3 anchors shall conform to Section 9-06.5(4) and be galvanized in accordance with AASHTO M 232, except that testing for embrittlement after galvanizing is not required. All other hardware shall conform to Section 9-06.5(1) and be galvanized in accordance with AASHTO M 232.

Grout for permanent installations of precast single slope barrier shall be in accordance with Section 6-02.3(20).

6-10.3 Construction Requirements

Single slope barrier shall be cast-in-place or slipformed, except when precast single slope barrier is specified in the Plans or approved by the Engineer. Concrete barrier installed in conjunction with light standard foundations and sign bridge foundations, regardless of the barrier shape, shall be cast-in-place using stationary forms.

Concrete barrier transition Type 2 to bridge f-shape shall be precast.

6-10.3(1) Precast Concrete Barrier

The fabrication plant for precast concrete barriers shall be approved by Contracting Agency prior to the use of barrier and the plant shall perform quality control testing and inspection on all barrier used by the Contracting Agency. The Contractor shall advise the Engineer of the production schedule for the fabrication of barrier.

Test results from the fabricators QC testing shall demonstrate compliance with sections 6-02.3(4)C consistency, 6-02.3(4)D temperature and time of placement, 6-02.3(2)A air content, and compressive strength. All tests will be conducted per section 6-02.3(5)D.

If self-compacting concrete (SCC) has been approved for use the requirements of Section 6-02.3(4)C consistency shall not apply. Self-compacting concrete (SCC) is concrete that is able to flow under its own weight and completely fill the formwork, even in the presence of dense reinforcement, without the need of any vibration, while maintaining homogeneity. When using SCC modified testing procedures for air content and compressive strength will be used. The modification shall be that molds will be filled completely in one continuous lift with out any rodding, vibration, tamping or other consolidation methods other than lightly taping around the exterior of the mold with a rubber mallet to allow entrapped air bubbles to escape. In addition the fabricators QC testing shall include Slump Flow Test results that do not indicate segregation. As part of the plants approval for use of SCC the plant fabricator shall cast one barrier and have that barrier sawed in half for examination by the Contracting Agency to ensure that segregation has not occurred.

The fabricators QC tester conducting the sampling and testing shall be qualified by ACI, Grade I to perform this work. The equipment used shall be calibrated/certified annually.

All test results and certifications shall be kept at the fabricator's facility for review by the Contracting Agency.

The Contracting Agency intends to perform Quality Assurance Inspection. This inspection is for the qualification of the plant QC process. This inspection shall not relieve the Contractor of any responsibility for identifying and replacing defective material and workmanship.

The concrete in precast barrier shall be Class 4000 and comply with the provisions of Section 6-02.3. No concrete barrier shall be shipped until test cylinders made of the same concrete and cured under the same conditions show the concrete has reached 4000 psi.

The Contractor may use Type III Portland cement, but shall bear any added cost.

Precast barrier shall be cast in steel forms. After release, the barrier shall be finished to an even, smooth, dense surface, free from any rock pockets or holes larger than $^1/_4$-inch across. Troweling shall remove all projecting concrete from the bearing surface.

Precast concrete barrier shall be cured in accordance with Section 6-02.3(25)D except that the barrier shall be cured in the forms until a rebound number test, or test cylinders which have been cured under the same conditions as the barrier, indicate the concrete has reached a compressive strength of a least 2500 psi. No additional curing is required once the barrier is removed from the forms.

The barrier shall be precast in sections as the Standard Plans require. All barrier in the same project (except end sections and variable length units needed for closure) shall be the same length. All barrier shall be new and unused. It shall be true to plan dimensions. The manufacturer shall be responsible for any damage or distortion that results from manufacturing.

Only one section less than 10-feet long may be used in any single run of precast barrier, and it must be at least 8-feet long. It may be precast or cast-in-place. Hardware identical to that used with other sections shall interlock such a section with adjacent precast sections.

Barrier connection voids for permanent installations of precast single slope barrier shall be filled with grout.

6-10.3(2) Cast-In-Place Concrete Barrier

Forms for cast-in-place concrete barrier, including traffic barrier, traffic-pedestrian barrier, and pedestrian barrier on bridges and related structures, shall be made of steel or exterior plywood coated with plastic. The Contractor may construct the barrier by the slip-form method.

The barrier shall be made of Class 4000 concrete that meets the requirements of Section 6-02, except that the fine aggregate gradation used for slip form barrier may be either Class 1 or 2. The Contractor may use Portland cement Type III at no additional expense to the Contracting Agency.

In addition to the steel reinforcing bar tying and bracing requirements specified in Section 6-02.3(24) C, the Contractor may also place small amounts of concrete to aid in holding the steel reinforcing bars in place. These small amounts of concrete shall be not more than two cubic feet in volume, and shall be spaced at a minimum of ten-foot intervals within the steel reinforcement cage. These small amounts of concrete shall be consolidated and shall provide two inches minimum clearance to the steel reinforcing bars on the outside face of the barrier. All spattered and excess mortar and concrete shall be removed from the steel reinforcing bars prior to slip-form casting.

Barrier expansion joints shall be spaced at 96-foot intervals, and dummy joints shall be spaced at 12-foot intervals unless otherwise specified in the contract.

Immediately after removing the forms, the Contractor shall complete any finishing work needed to produce a uniformly smooth, dense surface. The surface shall have no rock pockets and no holes larger than 1/4-inch across. The barrier shall be cured and finished in accordance with Section 6-02.3(11)A.

The maximum allowable deviation from a 10-foot straightedge held longitudinally on all surfaces shall be 1/4-inch. For single sloped barrier the maximum allowable deviation from a straightedge held along the vertical sloped face of the barrier shall be 1/4-inch.

At final acceptance of the project, the barrier shall be free from stains, smears, and any discoloration.

6-10.3(3) Removing and Resetting Permanent Concrete Barrier

The Contractor shall reset concrete barrier if the Plans or the Engineer require. If resetting is impossible immediately after removal, the Contractor shall store the barrier at Engineer-approved locations.

6-10.3(4) Joining Precast Concrete Barrier to Cast-In-Place Barrier

The Contractor may join segments of cast-in-place barrier to precast barrier where transitions, split barriers, or gaps shorter than 10-feet require it. At each joint of this type, the cast-in-place segment shall include hardware that ties both its ends to abutting precast sections.

6-10.3(5) Temporary Concrete Barrier

For temporary concrete barrier, the Contractor may use new or used precast barrier. This barrier shall comply with Standard Plan requirements and cross-sectional dimensions, except that: (1) it may be made in other lengths than those shown in the Standard Plan, and (2) it may have permanent lifting holes no larger than 4-inches in diameter or lifting loops. The word “temporary” shall be visibly stamped or stencil painted on each barrier segment.

If the contract calls for the removal and resetting of permanent barrier, and the permanent barrier is not required to remain in place until reset, the permanent barrier may be substituted for temporary concrete barrier and will not be stamped or stenciled "temporary". Any of the permanent barrier damaged during its use as temporary barrier will become the property of the contractor and be replaced with permanent barrier at no expense to the Contracting Agency when the permanent barrier is reset to its permanent location.

All barrier shall be in good condition, without cracks, chips, spalls, dirt, or traffic marks. If any barrier segment is damaged during or after placement, the Contractor, at no expense to the Contracting Agency, shall immediately repair it to the Engineer's satisfaction or replace it with an undamaged section.

As soon as the temporary barrier is no longer needed, the Contractor shall remove it from the project. Contracting Agency furnished barrier shall remain Contracting Agency property, and the Contractor shall deliver it to a stockpile site noted in the contract or to locations as approved by the Engineer. Contractor furnished barrier shall remain the property of the Contractor.

6-10.3(6) Placing Concrete Barrier

Precast concrete barrier shall rest on a paved foundation shaped to a uniform grade and section. The foundation surface shall meet this test for uniformity: When a 10-foot straightedge is placed on the surface parallel to the centerline for the barrier, the surface shall not vary more than 1/4-inch from the lower edge of the straightedge. If deviations exceed 1/4-inch, the Contractor shall correct them as required in Section 5-04.3(13).

The Contractor shall align the joints of precast segments so that they offset no more than 1/4-inch transversely and no more than 3/4-inch vertically. Grouting is not permitted, except as previously stated for single slope barrier. If foundation grade and section are acceptable, the Engineer may permit the Contractor to obtain vertical alignment of the barrier by shimming. Shimming shall be done with a polystyrene, foam pad (12 by 24-inches) under the end 12-inches of bearing surface.

Precast barrier shall be handled and placed with equipment that will not damage or disfigure it.

6-10.4 Measurement

Precast concrete barrier will be measured by the linear foot along its completed line and slope.

Temporary concrete barrier will be measured by the linear foot along the completed line and slope of the barrier, one time only for each setup of barrier protected area. Any intermediate moving or resetting will not be measured.

Cast-in-place concrete barrier will be measured by the linear foot along its completed line unless the contract specifies that it be measured per cubic yard for concrete Class 4000 and per pound for steel reinforcing bar (as required in Section 6-02.4).

Cast-in-place concrete barrier light standard section will be measured by the unit for each light standard section installed.

Removing and resetting existing permanent barrier will be measured by the linear foot and will be measured one time only for removing, storage, and resetting. No measure will be made for barrier that has been removed and reset for the convenience of the Contractor.

Concrete barrier transition Type 2 to bridge F-shape will be measured by the linear foot installed.

Single slope concrete barrier light standard foundation will be measured by the unit for each light standard foundation installed.

Traffic barrier, traffic pedestrian barrier, and pedestrian barrier will be measured as specified for cast-in-place concrete barrier.

6-10.5 Payment

Payment will be made in accordance with Section 1-04.1, for each of the following bid items that are included in the proposal:

"Precast Conc. Barrier Type ____", per linear foot.

"Cast-In-Place Conc. Barrier", per linear foot.

"Conc. Class 4000", per cubic yard.

"St. Reinf. Bar", per pound.

"Removing and Resetting Existing Permanent Barrier", per linear foot.

The unit contract price per linear foot for "Cast-In-Place Conc. Barrier" shall be full pay for excavation, forms, placement, special construction features, and all other materials, tools, equipment, and labor necessary to complete the work as specified; except that when the contract specifies, the unit contract price per cubic yard for "Conc. Class 4000" and the per pound for "St. Reinf. Bar" shall be full pay for excavation, forms, placement, special construction features, and all other materials, tools, equipment, and labor necessary to complete the work as specified.

"Traffic Barrier", per linear foot.

"Traffic Pedestrian Barrier", per linear foot.

"Pedestrian Barrier" per linear foot.

The unit contract price per linear foot for "Traffic Barrier", "Traffic Pedestrian Barrier", and "Pedestrian Barrier" shall be full pay for constructing the barrier on top of the bridge deck, and associated bridge approach slabs, curtain walls and wingwalls, excluding the steel reinforcing bars that extend from the bridge deck, bridge approach slab, curtain walls, and wingwalls.

"Single Slope Concrete Barrier", per linear foot.

The unit contract price per linear foot for "Single Slope Concrete Barrier" shall be full pay for either cast-in-place or precast single slope concrete barrier.

"Conc. Barrier Transition Type 2 to Bridge F-Shape", per linear foot.

The unit contract price per linear foot for "Conc. Barrier Transition Type 2 to Bridge F-Shape" shall be full pay for performing the work as specified, excluding bridge traffic barrier modifications necessary for this installation.

"Single Slope Conc. Barrier Light Standard Foundation", per each.

"Cast-In-Place Conc. Barrier Light Standard Section", per each.

"Temporary Conc. Barrier", per linear foot.

The unit contract price per linear foot for "Temporary Concrete Barrier" shall be full pay for all costs, including furnishing, installing, connecting, anchoring, maintaining, temporary storage, and final removal of the temporary barrier.

Payment for transition sections between different types of barrier shall be made at the unit contract price for the type of barrier indicated in the Plans for each transition section.

6-11 PRECAST CONCRETE RETAINING WALL STEMS

6-11.1 Description

The Contractor may construct Standard Plan Reinforced Concrete Retaining Walls Type 1, 2, 3, and 4 using precast concrete wall stems as specified herein.

6-11.1(1) Submittals

Before proceeding with construction of the retaining walls using precast concrete wall stems, the Contractor shall submit the following to the Engineer for approval in accordance with Section 6-02.3(16):

1. Working drawings for fabrication of the wall stems, showing dimensions, reinforcing steel, joint and joint filler details, lifting devices with the manufacturer's recommended safe working capacity, and material specifications.
2. Falsework plans for the erection of the wall stems showing dimensions, support points, support footing sizes, erection blockouts, member sizes, connections, and material specifications.
3. Calculations for the precast wall, the connection between the precast wall and the cast-in-place footing, and any modifications to the cast-in-place footing. Calculations shall be prepared by a professional civil engineer licensed in the state of Washington.

6-11.2 Materials

Concrete for the precast concrete wall stems shall meet all the requirements for concrete Class 4000 as stated in Section 6-02.3.

Concrete for the cast-in-place footing shall meet all the requirements for concrete Class 4000 as stated in Section 6-02.3.

6-11.3 Construction Requirements

The precast concrete wall stems shall be fabricated in accordance with the dimensions and details shown in the Plans, except as modified in the approved working drawings.

The precast concrete wall stems may be fabricated full height in 8-foot, 16-foot, 24-foot widths.

The precast concrete wall stems shall be constructed with a mating shear key between adjacent panels. The shear key shall have beveled corners and shall be $1\frac{1}{2}$-inches in thickness. The width of the shear key shall be $3\frac{1}{2}$-inches minimum and $5\frac{1}{2}$-inches maximum. The shear key shall be continuous and shall be of uniform width over the entire height of the wall stem.

Rolled on textured finishes shall not be used. Precast stem walls shall be cast in a vertical position if the Plans call for a form liner texture on both sides of the stem wall.

The precast wall panel shall be rigidly held in place during placement and curing of the footing concrete.

To ensure an even flow of concrete under and against the base of the wall, a form shall be placed parallel to the wall stem, above the footing, to allow a minimum 1-foot head to develop in the concrete during concrete placement.

The reinforcing steel shall be shifted to clear the erection blockouts in the wall stem by $1\frac{1}{2}$-inches minimum.

All panel joints shall be constructed with joint filler installed on the rear (backfill) side of the wall. The joint filler material shall extend from 2-feet below the final ground level in front of the wall to the top of the wall. The joint filler shall be a nonorganic flexible material and shall be installed to create a waterproof seal at panel joints.

The soil bearing pressure beneath the falsework supports for the wall stems shall not exceed the maximum design soil pressure shown in the Plans for the retaining wall.

The wall stem shall be placed a minimum of 1-inch into the footing to provide a shear key. The base of the stem shall be sloped $^1/_2$-inch per foot to facilitate proper concrete placement.

6-11.3(1) Tolerances

The construction tolerances for the precast retaining wall stems shall be:

Height	$\pm^1/_4$-inch
Width	$\pm^1/_4$-inch
Thickness	$^1/_4$-inch
	$+\!-^1/_8$-inch
Conc. cover for steel	$-^1/_8$-inch
reinforcing bar	$+^3/_8$-inch
Width of panel joints	$\pm^1/_4$-inch
Offset of panels (deviation from a straight line extending 5-feet on each side of panel joint)	$\pm^1/_4$-inch

6-11.4 Measurement

Measurement of the materials involved in constructing the precast concrete retaining wall stems and cast-in-place footing will be in accordance with Section 6-02.4 for the applicable related bid items of work involved in constructing Standard Plan Reinforced Concrete Retaining Walls Type 1, 2, 3, and 4.

6-11.5 Payment

All costs associated with constructing the retaining walls using precast concrete retaining wall stems shall be included in the unit contract prices for the applicable related bid items of work required for construction of Standard Plan Reinforced Concrete Retaining Walls Type 1, 2, 3, and 4.

6-12 NOISE BARRIER WALLS

6-12.1 Description

This work consists of constructing cast-in-place concrete, precast concrete, masonry, and timber noise barrier walls, including those shown in the Standard Plans.

6-12.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Portland Cement Concrete	9-03.1
Gravel Backfill	9-03.12
Premolded Joint Filler	9-04.1(2)
Bolts, Nuts, and Washers	9-06.5(1)
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Paints	9-08
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25

Other materials required shall be as specified in the Special Provisions.

6-12.3 Construction Requirements

6-12.3(1) Submittals

All noise barrier walls not constructed immediately adjacent to the roadway, and that require construction of access for work activities, shall have a noise barrier wall access plan. The Contractor shall submit the noise barrier wall access plan to the Engineer for approval in accordance with Section 6-01.9. The noise barrier wall access plan shall include, but not be limited to, the locations of access to the noise barrier wall construction sites, and the method, materials, and equipment used to construct the access, remove the access, and recontour and reseed the disturbed ground.

For construction of all noise barrier walls with shafts, the Contractor shall submit a shaft construction plan to the Engineer for approval in accordance with Section 6-01.9, including but not limited to the following information:

1. List and description of equipment to be used to excavate and construct the shafts, including description of how the equipment is appropriate for use in the expected subsurface conditions.
2. The construction sequence and order of shaft construction.
3. Details of shaft excavation methods, including methods to clean the shaft excavation.
4. Details and dimensions of the shaft, and casing if used.
5. The method used to prevent ground caving (temporary casing, slurry, or other means).
6. Details of concrete placement including procedures for deposit through a conduit, tremie, or pump.
7. Method and equipment used to install and support the steel reinforcing bar cage.

For construction of precast concrete noise barrier walls, the Contractor shall submit shop drawings for the precast concrete panels to the Engineer in accordance with Section 6-02.3(28)A. In addition to the items listed in Section 6-02.3(28)A, the precast concrete panel shop drawings shall include the following:

1. Construction sequence and method of forming the panels.
2. Details of additional reinforcement provided at lifting and support locations.
3. Method and equipment used to support the panels during storage, transporting, and erection.
4. Erection sequence, including the method of lifting the panels, placing and adjusting the panels to proper alignment and grade, and supporting the panels during bolting, grouting, and backfilling operations.

The Contractor shall not begin noise barrier wall construction activities, including access construction and precast concrete panel fabrication, until receiving the Engineer's approval of all appropriate and applicable submittals.

6-12.3(2) Work Access and Site Preparation

The Contractor shall construct work access in accordance with the work access plan as approved by the Engineer. The construction access roads shall minimize disturbance to the existing vegetation, especially trees. Only trees and shrubs in direct conflict with the approved construction access road alignment shall be removed. Only one access road into the noise barrier wall from the main roadway and one access road from the noise barrier wall to the main roadway shall be constructed at each noise barrier wall.

Existing vegetation that has been identified by the Engineer shall be protected in accordance with Sections 1-07.16 and 2-01, and the Special Provisions.

6-12.3(3) Shaft Construction

The Contractor shall excavate and construct the shafts in accordance with the shaft construction plan as approved by the Engineer.

The shafts shall be excavated to the required depth as shown in the Plans. The excavation shall be completed in a continuous operation using equipment capable of excavating through the type of material expected to be encountered.

If the shaft excavation is stopped, the Contractor shall secure the shaft by installing a safety cover over the opening. The Contractor shall ensure the safety of the shaft and surrounding soil and the stability of the side walls. A temporary casing, slurry, or other methods approved by the Engineer shall be used as necessary to ensure such safety and stability.

When caving conditions are encountered, the Contractor shall stop further excavation until implementing the method to prevent ground caving as specified in the shaft construction plan approved by the Engineer.

When obstructions are encountered, the Contractor shall notify the Engineer promptly. An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and man made objects) encountered during the shaft excavation operation, which prevents or hinders the advance of the shaft excavation. When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being is significantly reduced relative to the rate of advance for the rest of the shaft excavation, then the Contractor shall remove the obstruction under the provisions of Section 6-12.5 as supplemented in the Special Provisions. The method of removal of such obstructions, and the continuation of excavation shall be as proposed by the Contractor and approved by the Engineer.

The Contractor shall use appropriate means to clean the bottom of the excavation of all shafts. No more than two inches of loose or disturbed material shall be present at the bottom of the shaft just prior to beginning concrete placement.

The Contractor shall not begin placing steel reinforcing bars and concrete in the shaft until receiving the Engineer's approval of the shaft excavation.

The steel reinforcing bar cage shall be rigidly braced to retain its configuration during handling and construction. The Contractor shall not place individual or loose bars. The Contractor shall install the steel reinforcing bar cage as specified in the shaft construction plan as approved by the Engineer. The Contractor shall maintain the minimum concrete cover shown in the Plans.

If casings are used, the Contractor shall remove the casing during concrete placement. A minimum five feet head of concrete shall be maintained to balance soil and water pressure at the bottom of the casing. The casing shall be smooth. Where the top of the shaft is above the existing ground, the Contractor shall case the top of the hole prior to placing the concrete.

Concrete for shafts shall conform to Class 4000P. The Contractor shall place concrete in the shaft immediately after completing the shaft excavation and receiving the Engineer's approval of the excavation. The Contractor shall place the concrete in one continuous operation to the elevation shown in the Plans, using a method to prevent segregation of aggregates. The Contractor shall place the concrete as specified in the approved shaft construction plan. If water is present, concrete shall be placed in accordance with Section 6-02.3(6)B.

6-12.3(4) Trench, Grade Beam, or Spread Footing Construction

Where the noise barrier wall foundations exist below the existing ground line, excavation shall conform to Section 2-09.3(4), and to the limits and construction stages shown in the Plans. Foundation soils found to be unsuitable shall be removed and replaced in accordance with Section 2-09.3(1)C.

Where the noise barrier wall foundations exist above the existing ground line, the Contractor shall place and compact backfill material in accordance with Section 2-03.3(14)C.

Concrete for trench, grade beam, or spread footing foundations shall conform to Class 4000.

Cast-in-place concrete shall be formed, placed, and cured in accordance with Section 6-02, except that concrete for trench foundations shall be placed against undisturbed soil.

The excavation shall be backfilled in accordance with item 1 of the Compaction subsection of Section 2-09.3(1)E.

The steel reinforcing bar cage and the noise barrier wall anchor bolts shall be installed and rigidly braced prior to grade beam and spread footing concrete placement to retain their configuration during concrete placement. The Contractor shall not place individual or loose steel reinforcing bars and anchor bolts, and shall not install anchor bolts during or after concrete placement.

6-12.3(5) Cast-In-Place Concrete Panel Construction

Construction of cast-in-place concrete panels for noise barrier walls shall conform to Section 6-11.3(4). For noise barrier walls with traffic barrier, the construction of the traffic barrier shall also conform to Section 6-10.3(2).

The top of the cast-in-place concrete panels shall conform to the top of wall profile shown in the Plans. Where a vertical step is constructed to provide elevation change between adjacent panels, the dimension of the step shall be 2-feet. Each horizontal run between steps shall be a minimum of 48-feet.

6-12.3(6) Precast Concrete Panel Fabrication and Erection

The Contractor shall fabricate and erect the precast concrete panels in accordance with Section 6-02.3(28), and the following requirements:

1. Concrete shall conform to Class 4000.
2. Except as otherwise noted in the Plans and Special Provisions, all concrete surfaces shall receive a Class 2 finish in accordance with Section 6-02.3(14)B.
3. The precast concrete panels shall be cast in accordance with Section 6-02.3(28)B. The Contractor shall cast the precast concrete panels horizontally, with the traffic side surface cast against the form liner on the bottom. The Contractor shall fully support the precast concrete panel to avoid bowing and sagging surfaces.

   After receiving the Engineer's approval of the shop drawings, the Contractor shall cast one precast concrete panel to be used as the sample panel. The Contractor shall construct the sample panel in accordance with the procedure and details specified in the shop drawings approved by the Engineer. The Contractor shall make the sample panel available to the Engineer for approval.

   Upon receiving the Engineer's approval of the sample panel, the Contractor shall continue production of precast concrete panels for the noise barrier wall. All precast concrete panels will be evaluated against the sample panel for the quality of workmanship exhibited. The sample panel shall be retained at the fabrication site until all precast concrete panels have been fabricated and have received the Engineer's approval. After completing precast concrete panel fabrication, the Contractor may utilize the sample panel as a production noise barrier wall panel.
4. In addition to the fabrication tolerance requirements of Section 6-02.3(28)F, the precast concrete panels for noise barrier walls shall not exceed the following scalar tolerances:

   Length and Width: $\pm$ $^1/_8$-inch per five feet, not to exceed $^1/_4$-inch total.

   Thickness: $\pm$ $^1/_4$-inch.

   The difference obtained by comparing the measurement of the diagonal of the face of the panels shall not be greater than $^1/_2$-inch.

   Dimension tolerances for the traffic barrier portion of precast concrete panels formed with traffic barrier shapes shall conform to Section 6-10.3(2).
5. After erection, the precast concrete panels shall not exceed the joint space tolerances shown in the Plans. The panels shall not exceed $^3/_8$-inch out of plumb in any direction.

   The Contractor shall seal the joints between precast concrete panels with a backer rod and sealant system as specified. The Contractor shall seal both sides of the joint full length.

The top of precast concrete panels shall conform to the top of wall profile shown in the Plans. Where a vertical step is constructed to provide elevation change between adjacent panels, the dimension of the step shall be 2-feet. Each horizontal run between steps shall be a minimum of 48-feet.

6-12.3(7) Masonry Wall Construction

Construction requirements for masonry noise barrier wall panels shall be as specified in the Special Provisions.

6-12.3(8) Fabricating and Erecting Timber Noise Barrier Wall Panels

Construction requirements for timber noise barrier wall panels shall be as specified in the Special Provisions.

6-12.3(9) Access Doors and Concrete Landing Pads

The Contractor shall install access doors and door frames as shown in the Plans and Standard Plans. The Contractor shall install the access doors to open toward the roadway side. The door frames shall be set in place with grout conforming to Section 6-02.3(20), with the grout completely filling the void between the door frame and the noise barrier wall panel.

The Contractor shall apply two coats of paint, as specified in the Special Provisions, to all exposed metal surfaces of access doors and frames, except for stainless steel surfaces. Each coat shall be 3 mils minimum wet film thickness.

The Contractor shall construct a concrete landing pad on the roadway side of each access door location as shown in the Plans. The concrete shall conform to Section 6-02.3(2)B.

6-12.3(10) Finish Ground Line Dressing

The Contractor shall contour and dress the ground line on both sides of the noise barrier wall, providing the minimum cover over the foundation as shown in the Plans. The Contractor shall contour the ground adjacent to the barrier to ensure good drainage away from the barrier.

After the access roads are no longer needed for noise barrier wall construction activities, the Contractor shall restore the area to the original condition. The Contractor shall recontour the access roads to match into the surrounding ground and shall reseed all disturbed areas in accordance with the Section 8-01 and the Special Provisions, and the noise barrier wall access plan as approved by the Engineer.

6-12.4 Measurement

Noise barrier wall will be measured by the square foot area of one face of the completed wall panel in place. Except as otherwise noted, the bottom limit for measurement will be the top of the trench footing, spread footing, or shaft cap. For Noise Barrier Type 5, the bottom measurement limit will be the optional construction joint at the base of the traffic barrier. For Noise Barrier Type 7, the bottom measurement limit will be base of the traffic barrier. For Noise Barrier Types 8, 11, 12, 14, 15, and 20, the bottom measurement limit will be the base of the wall panel.

Noise barrier wall access door will be measured once for each access door assembly with concrete landing pad furnished and installed.

6-12.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Noise Barrier Wall Type __", per square foot.

The unit contract price per square foot for "Noise Barrier Wall Type __" shall be full pay for constructing the noise barrier walls as specified, including constructing and removing access roads, excavating and constructing foundations and grade beams, constructing cast-in-place concrete, and masonry wall panels, fabricating and erecting precast concrete, and timber wall panels, applying sealer, and contouring the finish ground line adjacent to the noise barrier walls.

"Noise Barrier Wall Access Door", per each.

The unit contract price per each for "Noise Barrier Wall Access Door" shall be full pay for furnishing and installing the access door assembly as specified, including painting the installed access door assembly and constructing the concrete landing pad.

6-13 STRUCTURAL EARTH WALLS

6-13.1 Description

This work consists of constructing structural earth walls (SEW).

6-13.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Portland Cement Concrete	9-03.1
Gravel Backfill	9-03.12(2)
Premolded Joint Filler	9-04.1(2)
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25

Other materials required shall be as specified in the Special Provisions.

6-13.3 Construction Requirements

Proprietary structural earth wall systems shall be as specified in the Special Provisions.

6-13.3(1) Quality Assurance

The structural earth wall manufacturer shall provide a qualified and experienced representative to resolve wall construction problems as approved by the Engineer. The structural earth wall manufacturer's representative shall be present at the beginning of wall construction activities, and at other times as needed throughout construction. Recommendations made by the structural earth wall manufacturer's representative and approved by the Engineer shall be followed by the Contractor.

The completed wall shall meet the following tolerances:

1. Deviation from the design batter and horizontal alignment, when measured along a ten foot straight edge, shall not exceed the following:
   a. Welded wire faced structural earth wall: 2-inches
   b. Precast concrete panel and concrete block faced structural earth wall: $^{3}/_{4}$-inch
2. Deviation from the overall design batter of the wall shall not exceed the following per ten feet of wall height:
   a. Welded wire faced structural earth wall: 1$^{1}/_{2}$-inches
   b. Precast concrete panel and concrete block faced structural earth wall: $^{1}/_{2}$-inch
3. The maximum outward bulge of the face between welded wire faced structural earth wall reinforcement layers shall not exceed two inches. The maximum allowable offset in any precast concrete facing panel joint shall be $^{3}/_{4}$-inch. The maximum allowable offset in any concrete block joint shall be $^{3}/_{4}$-inch.
4. The base of the structural earth wall excavation shall be within three inches of the staked elevations, unless otherwise approved by the Engineer.

5. The external structural earth wall dimensions shall be placed within two inches of that staked on the ground.
6. The backfill reinforcement layers shall be located horizontally and vertically within one inch of the locations shown in the structural earth wall working drawings as approved by the Engineer.

At least five working days prior to the Contractor beginning any structural earth wall work at the site, a structural earth wall preconstruction conference shall be held to discuss construction procedures, personnel, and equipment to be used, and other elements of structural earth wall construction. Those attending shall include:

1. (representing the Contractor) The superintendent, on site supervisors, and all foremen in charge of excavation, leveling pad placement, concrete block and soil reinforcement placement, and structural earth wall backfill placement and compaction.
2. (representing the Structural Earth Wall Manufacturer) The qualified and experienced representative of the structural earth wall manufacturer as specified at the beginning of this Section.
3. (representing the Contracting Agency) The Project Engineer, key inspection personnel, and representatives from the WSDOT Construction Office and Materials Laboratory Geotechnical Services Branch.

6-13.3(2) Submittals

The Contractor, or the supplier as the Contractor's agent, shall furnish to the Engineer a Manufacturer's Certificate of Compliance in accordance with Section 1-06.3, certifying that the structural earth wall materials conform to the specified material requirements. This includes providing a Manufacturer's Certificate of Compliance for all concrete admixtures, cement, fly ash, steel reinforcing bars, reinforcing strips, reinforcing mesh, tie strips, fasteners, welded wire mats, backing mats, construction geotextile for wall facing, drainage geosynthetic fabric, block connectors, and joint materials. The Manufacturer's Certificate of Compliance for geogrid reinforcement shall include the information specified in Section 9-33.4(4) for each geogrid roll, and shall specify the geogrid polymer types for each geogrid roll.

A copy of all test results, performed by the Contractor or the Contractor's supplier, which are necessary to assure compliance with the specifications, shall submitted to the Engineer along with each Manufacturer's Certificate of Compliance.

Before fabrication, the Contractor shall submit a field construction manual for the structural earth walls, prepared by the wall manufacturer, to the Engineer for approval in accordance with Section 6-01.9. This manual shall provide step-by-step directions for construction of the wall system.

The Contractor, or the supplier as the Contractor's agent, shall submit detailed design calculations and working drawings to the Engineer for approval in accordance with Section 6-01.9.

The design calculation and working drawing submittal shall include detailed design calculations and all details, dimensions, quantities, and cross-sections necessary to construct the wall. The calculations shall include a detailed explanation of any symbols and computer programs used in the design of the walls. All computer output submitted shall be accompanied by supporting hand calculations detailing the calculation process.

The design calculations shall be based on the current AASHTO Standard Specifications for Highway Bridges including current interims, and also based on the following:

1. The factor of safety for overturning and sliding are 2.0 and 1.5 respectively for AASHTO Load Group I, and 1.5 and 1.1 respectively for AASHTO Load Group VII.
2. The wall surcharge conditions (backfill slope) shown in the Plans.
3. If a highway is adjacent to and on top of the wall, a two foot surcharge shall be used in the design.
4. If the Plans detail a traffic barrier on top of the wall, the barrier and wall shall be capable of resisting a 10,000 pound horizontal load applied at the top of the barrier.
5. The geotechnical design parameters for the wall shall be as specified in the Special Provisions.

A minimum of six sets of working drawings shall be fully detailed and shall include, but not be limited to, the following items:

1. A plan and elevation sheet or sheets for each wall, containing the following:
    a. An elevation view of the wall that includes the following:
        i. the elevation at the top of the wall, at all horizontal and vertical break points, and at least every 50-feet along the wall;
        ii. elevations at the base of welded wire mats or the top of leveling pads and foundations, and the distance along the face of the wall to all steps in the welded wire mats, foundations and leveling pads;
        iii. the designation as to the type of panel, block, or module;
        iv. the length, size, and number of geogrids or mesh or strips, and the distance along the face of the wall to where changes in length of the geogrids or mesh or strips occur; or
        v. the length, size, and wire sizes and spacing of the welded wire mats and backing mats, and the distance along the face of the wall to where changes in length, size, and wire sizes and spacing of the welded wire mats and backing mats occur; and
        vi. the location of the original and final ground line.
    b. A plan view of the wall that indicates the offset from the construction centerline to the face of the wall at all changes in horizontal alignment; the limit of the widest module, geogrid, mesh, strip or welded wire mat, and the centerline of any drainage structure or drainage pipe which is behind or passes under or through the wall.
    c. General notes, if any, required for design and construction of the wall.
    d. All horizontal and vertical curve data affecting wall construction.
    e. A listing of the summary of quantities provided on the elevation sheet of each wall for all items including incidental items.
    f. Cross-section showing limits of construction. In fill sections, the cross-section shall show the limits and extent of select granular backfill material placed above original ground.
    g. Limits and extent of reinforced soil volume.

2. All details including steel reinforcing bar bending details. Bar bending details shall be in accordance with Section 9-07.1.
3. All details for foundations and leveling pads, including details for steps in the foundations or leveling pads, as well as allowable and actual maximum bearing pressures for AASHTO Load Groups I and VII.
4. All modules and facing elements shall be detailed. The details shall show all dimensions necessary to construct the element, all steel reinforcing bars in the element, and the location of reinforcement element attachment devices embedded in the precast concrete facing panel or concrete block.
5. All details for construction of the wall around drainage facilities, sign, signal, luminaire, and noise barrier wall foundations, and structural abutment and foundation elements shall be clearly shown.
6. All details for connections to traffic or pedestrian barriers, coping, parapets, noise barrier walls, and attached lighting shall be shown.
7. All details for the traffic or pedestrian barrier attached to the top of the wall (if shown in the Plans) including interaction with bridge approach slabs.

The Contractor shall not begin wall construction (including precast concrete facing panel or block fabrication) until receiving the Engineer's written approval of the material certifications and test results, design calculations and working drawing submittals.

6-13.3(3) Excavation and Foundation Preparation

Excavation shall conform to Section 2-09.3(4) and to the limits and construction stages shown in the Plans. Foundation soils found to be unsuitable shall be removed and replaced in accordance with Section 2-09.3(1)C. The foundation for the structure shall be graded level for a width equal to or exceeding the length of reinforcing as shown in the structural earth wall working drawings as approved by the Engineer and, for walls with geogrid reinforcing, in accordance with Section 2-12.3. Prior to wall construction, the foundation, if not in rock, shall be compacted as approved by the Engineer.

At the foundation level of the bottom course of precast concrete facing panels and concrete blocks, an unreinforced concrete leveling pad shall be provided as shown in the Plans. The leveling pad shall be cured a minimum of 12 hours and have a minimum compressive strength of 1500 psi before placement of the precast concrete facing panels or concrete blocks.

6-13.3(4) Precast Concrete Facing Panel and Concrete Block Fabrication

Concrete for precast concrete facing panels shall meet the following requirements:

1. Have a minimum 28 day compressive strength of 4,000 pounds per square inch, unless otherwise specified in the Special Provisions for specific proprietary wall systems.
2. Contain a water-reducing admixture meeting AASHTO M 194 Type A, D, F, or G.
3. Be air-entrained, 6 percent ± 1$^1/_2$ percent.
4. Have a maximum slump of four inches, or six inches if a Type F or G water reducer is used.

Concrete for dry cast concrete blocks shall meet the following requirements:

1. Have a minimum 28 day compressive strength of 4,000 psi.
2. Conform to ASTM C 1372, except as otherwise specified.
3. The lot of blocks produced for use in this project shall conform to the following freeze-thaw test requirements when tested in accordance with ASTM C 1262. Minimum acceptable performance shall be defined as weight loss at the conclusion of 150 freeze-thaw cycles not exceeding one percent of the block's initial weight for a minimum of four of the five block specimens tested.
4. The concrete blocks shall have a maximum water absorption of one percent above the water absorption content of the lot of blocks produced and successfully tested for the freeze-thaw test specified in item 3 above.

Precast concrete facing panels and concrete blocks will be accepted based on successful compressive strength tests, WSDOT "Approved for Shipment" stamp or tag, and visual inspection at the jobsite. The precast concrete facing panels and concrete blocks shall be considered acceptable regardless of curing age when compressive test results indicate that the compressive strength conforms to the 28-day requirements and when the visual inspection is satisfactorily completed. Fabrication of precast concrete facing panels and blocks shall conform to Section 6-02.3(28). Testing of dry cast concrete blocks shall conform to ASTM C 140.

All precast concrete facing panels shall be five feet square, except:

1. for partial panels at the top, bottom, and ends of the wall, and
2. as otherwise shown in the Plans.

All precast concrete facing panels shall be manufactured within the following tolerances:

1. All dimensions ± $^{3}/_{16}$-inch.
2. Squareness, as determined by the difference between the two diagonals, shall not exceed $^{1}/_{2}$-inch.
3. Surface defects on smooth formed surfaces measured on a length of 5-feet shall not exceed $^{1}/_{8}$-inch. Surface defects on textured-finished surfaces measured on a length of five feet shall not exceed $^{5}/_{16}$-inch.

All concrete blocks shall be manufactured within the following tolerances:

1. Vertical dimensions shall be ± $^{1}/_{16}$-inch of the plan dimension, and the rear height shall not exceed the front height.
2. The dimensions of the grooves in the top and bottom faces of the concrete blocks shall be formed within the tolerances specified by the proprietary wall manufacturer, for the fit required for the block connectors.
3. All other dimensions shall be ± $^{1}/_{4}$-inch of the plan dimension.

Tie attachment devices, except for geosynthetic reinforcement, shall be set in place to the dimensions and tolerances shown in the Plans prior to casting.

The forms forming precast concrete facing panels, including the forms for loop pockets and access pockets, and the forms forming the concrete blocks, shall be removed in accordance with the recommendations of the wall manufacturer, without damaging the concrete.

The concrete surface for the precast concrete facing panel shall have the finish shown in the Plans for the front face and an unformed finish for the rear face. The rear face of the precast concrete facing panel shall be roughly screeded to eliminate open pockets of aggregate and surface distortions in excess of 1/4-inch.

The concrete surface for the front face of the concrete block shall be flat, and shall be a conventional “split face” finish in accordance with the wall manufacturer’s specifications. The concrete surface of all other faces shall be Class 2 in accordance with Section 6-02.3(14)B. The finish and appearance of the concrete blocks shall also conform to ASTM C 1372. The color of the concrete block shall be concrete gray, unless otherwise shown in the Plans.

The date of manufacture, production lot number, and the piece-mark, shall be clearly marked on the rear face of each precast concrete facing panel, and marked or tagged on each pallet of concrete blocks.

All precast concrete facing panels and concrete blocks shall be handled, stored, and shipped in accordance with Sections 6-02.3(28)G and 6-02.3(28)H to prevent chipping, cracks, fractures, and excessive bending stresses.

Precast concrete facing panels in storage shall be supported on firm blocking located immediately adjacent to tie strips to avoid bending the tie strips.

6-13.3(5) Precast Concrete Facing Panel and Concrete Block Erection

The precast concrete facing panels shall be placed vertically. During erection, precast concrete facing panels shall be handled by means of a lifting device set into the upper edge of the panels.

Concrete blocks shall be erected in a running bond fashion in accordance with the wall manufacturer’s field construction manual, and may be placed by hand. The top surface of each course of concrete blocks, including all pockets and recesses, shall be cleaned of backfill and all extraneous materials prior to connecting the reinforcing strips or geosynthetic reinforcing, and placing the next course of concrete blocks. Concrete blocks receiving geosynthetic reinforcement shall be connected as specified in the Special Provisions. Cap block top courses shall be bonded to the lower course of concrete blocks as specified below. All other concrete blocks shall be connected with block connectors or pins placed into the connector slots.

Precast concrete facing panels and concrete blocks shall be placed in successive horizontal lifts as backfill placement proceeds in the sequence shown in the structural earth wall working drawings as approved by the Engineer.

External bracing is required for the initial lift for precast concrete facing panels.

As backfill material is placed behind the precast concrete facing panels, the panels shall be maintained in vertical position by means of temporary wooden wedges placed in the joint at the junction of the two adjacent panels on the external side of the wall.

Reinforcing shall be placed normal to the face of the wall, unless otherwise shown in the Plans or directed by the Engineer. Prior to placement of the reinforcing, backfill shall be compacted.

Geosynthetic reinforcing shall be placed in accordance with Section 2-12.3 and as follows:

1. The Contractor shall stretch out the geosynthetic in the direction perpendicular to the wall face to remove all slack and wrinkles, and shall hold the geosynthetic in place with soil piles or other methods as recommended by the geosynthetic manufacturer, before placing backfill material over the geosynthetic to the specified cover.
2. The geosynthetic reinforcement shall be continuous in the direction perpendicular to the wall face from the back face of the concrete panel to the end of the geosynthetic or to the last geogrid node at the end of the specified reinforcement length. Geosynthetic splices parallel to the wall face will not be allowed.

At the completion of each course of concrete blocks and prior to installing any block connectors or geosynthetic reinforcement at this level, the Contractor shall check the blocks for level placement in all directions, and shall adjust the blocks by grinding or rear face shimming, or other method as recommended by the structural earth wall manufacturer's representative and as approved by the Engineer, to bring the blocks into a level plane.

For concrete block wall systems receiving a cap block top course, the cap blocks shall be bonded to the lower course with mortar, or with an adhesive capable of bonding the concrete block courses together.

6-13.3(6) Welded Wire Faced Structural Earth Wall Erection

The Contractor shall erect the welded wire wall reinforcement in accordance with the wall manufacturer's field construction manual and as approved by the Engineer. Construction geotextile for wall facing shall be placed between the backfill material within the reinforced zone and the coarse granular material immediately behind the welded wire wall facing, as shown in the Plans and the structural earth wall working drawings as approved by the Engineer.

6-13.3(7) Backfill

Backfill placement shall closely follow erection of each course of welded wire mats and backing mats, precast concrete facing panels, or concrete blocks. Backfill shall be placed in such a manner as to avoid any damage or disturbance to the wall materials or misalignment of the welded wire mats and backing mats, precast concrete facing panels, or concrete blocks. Backfill shall be placed in a manner that segregation does not occur.

The Contractor shall place wall backfill over geosynthetic reinforcement, or construction geotextile for wall facing, in accordance with Section 2-12.3 and as follows:

1. The Contractor shall ensure that six inches minimum of backfill shall be between the geogrid reinforcement, or construction geotextile for wall facing, and any construction vehicle or equipment tires or tracks at all times.

Misalignment or distortion of the precast concrete facing panels or concrete blocks due to placement of backfill outside the limits of this specification shall be corrected in a manner as approved by the Engineer.

The moisture content of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer of material. The moisture content of all backfill material shall conform to Sections 2-03.3(14)C and 2-03.3(14)D.

Backfill shall be compacted in accordance with Method C of Section 2-03.3(14)C, except as follows:

1. The maximum lift thickness after compaction shall not exceed ten inches.
2. The Contractor shall decrease this lift thickness, if necessary, to obtain the specified density.
3. The Contractor shall not use sheepsfoot rollers or rollers with protrusions for compacting backfill reinforced with geosynthetic layers, or for compacting the first lift of backfill above the construction geosynthetic for wall facing for each layer of welded wire mats. Rollers shall have sufficient capacity to achieve compaction without causing distortion to the face of the wall in accordance with the tolerances specified in Section 6-13.3(1).
4. The Contractor shall compact the zone within three feet of the back of the wall facing panels without causing damage to or distortion of the wall facing elements (welded wire mats, backing mats, construction geotextile for wall facing, precast concrete facing panels, and concrete blocks) by using light mechanical tampers as approved by the Engineer. No soil density tests will be taken within this area.
5. For wall systems with geosynthetic reinforcement, the minimum compacted backfill lift thickness of the first lift above each geosynthetic reinforcement layer shall be six inches.

At the end of each day's operation, the Contractor shall shape the last level of backfill to permit runoff of rainwater away from the wall face. In addition, the Contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

Wall materials damaged or disturbed during backfill placement shall be either removed and replaced, or adjusted and repaired, by the Contractor as approved by the Engineer at no additional expense to the Contracting Agency.

6-13.3(8) Guardrail Placement

Where guardrail posts are required, the Contractor shall not begin installing guardrail posts until completing the structural earth wall to the top of wall elevation shown in the Plans. The Contractor shall install the posts in a manner that prevents movement of the precast concrete facing panels or concrete blocks, and prevents ripping, tearing, or pulling of the wall reinforcement.

The Contractor may cut welded wire reinforcement of welded wire faced structural earth walls to facilitate placing the guardrail posts, but only in the top two welded wire reinforcement layers and only with the approval of the Engineer in a manner that prevents bulging of the wall face and prevents ripping or pulling of the welded wire reinforcement. Holes through the welded wire reinforcement shall be the minimum size necessary for the post. The Contractor shall demonstrate to the Engineer prior to beginning guardrail post installation that the installation method will not rip, tear, or pull the wall reinforcement.

The Contractor shall place guardrail posts between the reinforcing strips, reinforcing mesh, and tie strips of the non-geosynthetic reinforced precast concrete panel or concrete block faced structural earth walls. Holes through the reinforcement of geosynthetic reinforced walls, if necessary, shall be the minimum size necessary for the guardrail post.

6-13.3(9) SEW Traffic Barrier and SEW Pedestrian Barrier

SEW traffic barrier and SEW pedestrian barrier shall be constructed in accordance with Sections 6-02.3(11)A and 6-10.3(2), and the details in the Plans and in the structural earth wall working drawings as approved by the Engineer.

6-13.4 Measurement

Structural earth wall will be measured by the square foot of completed wall in place. The bottom limits for vertical measurement will be the bottom of the bottom mat, for welded wire faced structural earth walls, or the top of the leveling pad (or bottom of wall if no leveling pad is present) for precast concrete panel or concrete block faced structural earth walls. The top limit for vertical measurement will be the top of wall as shown in the Plans. The horizontal limits for measurement are from the end of the wall to the end of the wall.

Backfill for structural earth wall including haul will be measured by the cubic yard in place determined by the limits shown in the Plans.

SEW traffic barrier, and SEW pedestrian barrier will be measured as specified in Section 6-10.4 for cast-in-place concrete barrier.

Structure excavation Class B, structure excavation Class B including haul, and shoring or extra excavation Class B, will be measured in accordance with Section 2-09.4.

6-13.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Structural Earth Wall", per square foot.

All costs in connection with furnishing materials for, and constructing, structural earth walls, including constructing leveling pads when specified, shall be included in the unit contract price per square foot for "Structural Earth Wall".

"Backfill for Structural Earth Wall Incl. Haul", per cubic yard.

All costs in connection with furnishing and placing backfill for structural earth wall, including hauling and compacting the backfill, and furnishing and placing the wall facing backfill for welded wire faced structural earth walls, shall be included in the unit contract price per cubic yard for "Backfill for Structural Earth Wall Incl. Haul".

"SEW Traffic Barrier", per linear foot.

"SEW Pedestrian Barrier", per linear foot.

The unit contract price per linear foot for "SEW ___ Barrier" shall be full pay for constructing the barrier on top of the structural earth wall, except that when these bid items are not included in the proposal, all costs in connection with performing the work as specified shall be included in the unit contract price per square foot for "Structural Earth Wall".

"Structure Excavation Class B", per cubic yard.

"Structure Excavation Class B Incl. Haul", per cubic yard.

"Shoring Or Extra Excavation Class B", per square foot.

6-14 GEOSYNTHETIC RETAINING WALLS

6-14.1 Description

This work consists of constructing geosynthetic retaining walls, including those shown in the Standard Plans.

6-14.2 Materials

Materials shall meet the requirements of the following sections:

Gravel Borrow For Geosynthetic Retaining Wall	9-03.14(4)
Construction Geosynthetic	9-33

The requirements specified in Section 2-12.2 for geotextile shall also apply to geosynthetic and geogrid materials used for permanent and temporary geosynthetic retaining walls.

Other materials required shall be as specified in the Special Provisions.

6-14.3 Construction Requirements

Temporary geosynthetic retaining walls are defined as those walls and wall components constructed and removed or abandoned before the physical completion date of the project or as shown in the Plans. All other geosynthetic retaining walls shall be considered as permanent.

6-14.3(1) Quality Assurance

The Contractor shall complete the base of the retaining wall excavation to within plus or minus three inches of the staked elevations unless otherwise directed by the Engineer. The Contractor shall place the external wall dimensions to within plus or minus two inches of that staked on the ground. The Contractor shall space the reinforcement layers vertically and place the overlaps to within plus or minus one inch of that shown in the Plans.

The completed wall(s) shall meet the following tolerances:

	Permanent Wall	Temporary Wall
Deviation from the designbatter and horizontal alignment for the face when measured along a ten foot straight edge at the midpoint of each wall layer shall not exceed:	3-inches	5-inches
Deviation from the overall design batter per ten feet of wall height shall not exceed:	2-inches	3-inches
Maximum outward bulge of the face between backfill reinforcement layers shall not exceed:	4-inches	6-inches

6-14.3(2) Submittals

A minimum of 14 calendar days prior to beginning construction of each wall the Contractor shall submit detailed plans for each wall in accordance with Section 6-01.9. As a minimum, the submittals shall include the following:

1. Detailed wall plans showing the actual lengths proposed for the geosynthetic reinforcing layers and the locations of each geosynthetic product proposed for use in each of the geosynthetic reinforcing layers.

2. The Contractor's proposed wall construction method, including proposed forming systems, types of equipment to be used and proposed erection sequence.
3. Manufacturer's Certificate of Compliance, samples of the retaining wall geosynthetic and sewn seams for the purpose of acceptance as specified.
4. Details of geosynthetic retaining wall corner construction, including details of the positive connection between the wall sections on both sides of the corner.
5. Details of terminating a top layer of retaining wall geosynthetic and backfill due to a changing retaining wall profile.

Approval of the Contractor's proposed wall construction details and methods shall not relieve the Contractor of their responsibility to construct the walls in accordance with the requirements of these Specifications.

6-14.3(3) Excavation and Foundation Preparation

Excavation shall conform to Section 2-09.3(4), and to the limits and construction stages shown in the Plans. Foundations soils found to be unsuitable shall be removed and replaced in accordance with Section 2-09.3(1)C.

The Contractor shall direct all surface runoff from adjacent areas away from the retaining wall construction site.

6-14.3(4) Erection and Backfill

The Contractor shall begin wall construction at the lowest portion of the excavation and shall place each layer horizontally as shown in the Plans. The Contractor shall complete each layer entirely before beginning the next layer.

Geotextile splices shall consist of a sewn seam or a minimum 1'-0" overlap. Geogrid splices shall consist of adjacent geogrid strips butted together and fastened using hog rings, or other methods approved by the Engineer, in such a manner to prevent the splices from separating during geogrid installation and backfilling. Splices exposed at the wall face shall prevent loss of backfill material through the face. The splicing material exposed at the wall face shall be as durable and strong as the material to which the splices are tied. The Contractor shall offset geosynthetic splices in one layer from those in the other layers such that the splices shall not line up vertically. Splices parallel to the wall face will not be allowed, as shown in the Plans.

The Contractor shall stretch out the geosynthetic in the direction perpendicular to the wall face to ensure that no slack or wrinkles exist in the geosynthetic prior to backfilling.

For geogrids, the length of the reinforcement required as shown in the Plans shall be defined as the distance between the geosynthetic wrapped face and the last geogrid node at the end of the reinforcement in the wall backfill.

The Contractor shall place fill material on the geosynthetic in lifts such that six inches minimum of fill material is between the vehicle or equipment tires or tracks and the geosynthetic at all times. The Contractor shall remove all particles within the backfill material greater than three inches in size. Turning of vehicles on the first lift above the geosynthetic will not be permitted. The Contractor shall not end dump fill material directly on the geosynthetic without the prior approval of the Engineer.

Should the geosynthetic be damaged or the splices disturbed, the backfill around the damaged or displaced area shall be removed and the damaged strip of geosynthetic replaced by the Contractor at no expense to the Contracting Agency.

The Contractor shall use a temporary form system to prevent sagging of the geosynthetic facing elements during construction. A typical example of a temporary form system and sequence of wall construction required when using this form are detailed in the Plans. Soil piles or the geosynthetic manufacturer's recommended method, in combination with the forming system shall be used to hold the geosynthetic in place until the specified cover material is placed.

The Contractor shall place and compact the wall backfill in accordance with the wall construction sequence detailed in the Plans and Method C of Section 2-03.3(14)D, except as follows:

1. The maximum lift thickness after compaction shall not exceed ten inches
2. The Contractor shall decrease this lift thickness, if necessary, to obtain the specified density.
3. Rollers shall have sufficient capacity to achieve compaction without causing distortion to the face of the wall in accordance with Section 6-14.3(1).
4. The Contractor shall not use sheepsfoot rollers or rollers with protrusions.
5. The Contractor shall compact the zone within three feet of the back of the wall facing panels without causing damage to or distortion of the wall facing elements (welded wire mats, backing mats, construction geotextile for wall facing, precast concrete facing panels, and concrete blocks) by using light mechanical tampers as approved by the Engineer. No soil density tests will be taken within this area.
6. For wall systems with geosynthetic reinforcement, the minimum compacted backfill lift thickness of the first lift above each geosynthetic reinforcement layer shall be six inches.

The Contractor shall construct wall corners at the locations shown in the Plans, and in accordance with the wall corner construction sequence and method submitted by the Contractor and approved by the Engineer. Wall angle points with an interior angle of less than 150 degrees shall be considered to be a wall corner. The wall corner shall provide a positive connection between the sections of the wall on each side of the corner such that the wall backfill material cannot spill out through the corner at any time during the design life of the wall. The Contractor shall construct the wall corner such that the wall sections on both sides of the corner attain the full geosynthetic layer embedment lengths shown in the Plans.

Where required by retaining wall profile grade, the Contractor shall terminate top layers of retaining wall geosynthetic and backfill in accordance with the method submitted by the Contractor and approved by the Engineer. The end of each layer at the top of the wall shall be constructed in a manner that prevents wall backfill material from spilling out the face of the wall throughout the life of the wall. If the profile of the top of the wall changes at a rate of 1:1 or steeper, this change in top of wall profile shall be considered to be a corner.

6-14.3(5) Guardrail Placement

The Contractor shall install guardrail posts as shown in the Plans after completing the wall, but before the permanent facing is installed. The Contractor shall install the posts in a manner that prevents bulging of the wall face and prevents ripping, tearing, or pulling of the geosynthetic reinforcement. Holes through the geosynthetic reinforcement shall be the minimum size necessary for the post. The Contractor shall demonstrate to the Engineer prior to beginning guardrail post installation that the installation method will not rip, tear, or pull the geosynthetic reinforcement.

6-14.3(6) Permanent Facing

The Contractor shall apply a permanent facing to the surface of all permanent geosynthetic retaining walls as shown in the Plans. Shotcrete facing, if shown in the Plans, shall conform to Section 6-18. Concrete fascia panel, if shown in the Plans, shall conform to Section 6-15.3(9).

6-14.3(7) Geosynthetic Retaining Wall Traffic Barrier and Geosynthetic Retaining Wall Pedestrian Barrier

Geosynthetic wall traffic barrier (single slope and f-shape) and geosynthetic retaining wall pedestrian barrier shall be constructed in accordance with Sections 6-02.3(11)A and 6-10.3(2), and the details in the Plans.

6-14.4 Measurement

Permanent geosynthetic retaining wall and temporary geosynthetic retaining wall will be measured by the square foot of face of completed wall.

Borrow for geosynthetic retaining wall backfill will be measured as specified in Section 2-03.4.

Shotcrete facing and concrete fascia panel will be measured by the square foot surface area of the completed facing or fascia panel, measured to the neat lines of the facing or panel as shown in the Plans.

Geosynthetic wall single slope traffic barrier, geosynthetic wall f-shape traffic barrier, and geosynthetic retaining wall pedestrian barrier will be measured as specified in Section 6-10.4 for cast-in-place concrete barrier.

Structure excavation Class B, structure excavation Class B including haul, and shoring or extra excavation Class B, will be measured in accordance with Section 2-09.4.

6-14.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Geosynthetic Retaining Wall", per square foot.

"Temporary Geosynthetic Retaining Wall", per square foot.

All costs in connection with constructing the temporary or permanent geosynthetic retaining wall as specified shall be included in the unit contract price per square foot for "Geosynthetic Retaining Wall" and "Temporary Geosynthetic Retaining Wall", including compaction of the backfill material and furnishing and installing the temporary forming system.

"Borrow for Geosynthetic Wall Incl. Haul", per ton or per cubic yard.

All costs in connection with furnishing and placing backfill material for temporary or permanent geosynthetic retaining walls as specified shall be included in the unit contract price per ton or per cubic yard for "Borrow for Geosynthetic Wall Incl. Haul".

"Concrete Fascia Panel", per square foot.

All costs in connection with constructing the concrete fascia panels as specified shall be included in the unit contract price per square foot for "Concrete Fascia Panel", including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified).

Shotcrete facing will be paid for in accordance with Section 6-18.5.

"Geosynthetic Wall Single Slope Traffic Barrier", per linear foot.

"Geosynthetic Wall F-Shape Traffic Barrier", per linear foot.

"Geosynthetic Retaining Wall Pedestrian Barrier", per linear foot.

The unit contract price per linear foot for "Geosynthetic Wall Single Slope Traffic Barrier", "Geosynthetic Wall F-Shape Traffic Barrier", and "Geosynthetic Retaining Wall Pedestrian Barrier" shall be full pay for constructing the barrier on top of the geosynthetic retaining wall.

"Structure Excavation Class B", per cubic yard.

"Structure Excavation Class B Incl. Haul", per cubic yard.

"Shoring Or Extra Excavation Class B", per square foot.

6-15 SOIL NAIL WALLS

6-15.1 Description

This work consists of constructing soil nail walls.

6-15.2 Materials

Materials shall meet the requirements of the following section:

Prefabricated Drainage Mat 9-33.2(3)

Other materials required, including materials for soil nails, shall be as specified in the Special Provisions.

6-15.3 Construction Requirements

6-15.3(1) General Description

Soil nailing shall consist of excavating to the layer limits shown in the Plans, drilling holes at the specified angle into the native material, placing and grouting epoxy coated or encapsulated steel reinforcing bars (soil nails) in the drilled holes, placing prefabricated drainage material and steel reinforcement, and applying a shotcrete facing over the steel reinforcement. After completing the wall to full height, the Contractor shall construct the concrete fascia panels as shown in the Plans.

All proprietary items used in the soil nailed structure shall be installed in accordance with the manufacturer's recommendations. In the event of a conflict between the manufacturer's recommendations and these specifications, these specifications shall prevail.

6-15.3(2) Contractor's Experience Requirements

The Contractor or Subcontractor performing this work shall have completed at least five projects, within the last five years, involving construction of retaining walls using soil nails or ground anchors or shall have completed the construction of two or more projects totaling at least 15,000 square feet of retaining wall with a minimum total of 500 soil nails or ground anchors.

The Contractor shall assign an engineer with at least three years of experience in the design and construction of permanently anchored or nailed structures to supervise the work. The Contractor shall not use consultants or manufacturer's representatives in order to meet the requirements of this section. Drill operators and on-site supervisors shall have a minimum of one year experience installing permanent soil nails or ground anchors.

Contractors or Subcontractors that are specifically prequalified in Class 36 work will be considered to have met the above experience requirements.

6-15.3(3) Submittals

Work shall not begin on any soil nail wall system until the Engineer has approved all of the required submittals. The Contractor shall submit the following information in accordance with Section 6-01.9 not less than 30 calendar days prior to the start of wall excavation.

1. A brief description of each project satisfying the Contractors Experience Requirements with the Owner's name and current phone number (this item is not required if the Contractor or Subcontractor is prequalified in Class 36).

2. A list identifying the following personnel assigned to this project and their experience with permanently anchored or nailed structures:
   a. Supervising Engineer
   b. Drill Operators
   c. On-site Supervisors who will be assigned to the project.
3. The proposed detailed construction procedure that includes:
   a. Proposed method(s) of excavation of the soil and/or rock.
   b. A plan for the removal and control of groundwater encountered during excavation, drilling, and other earth moving activities. Include a list of the equipment used to remove and control groundwater.
   c Proposed drilling methods and equipment.
   d. Proposed hole diameter(s).
   e. Proposed method of soil nail installation.
   f. Grout mix design and procedures for placing the grout.
   g. Shotcrete mix design with compressive strength test results.
   h. Procedures for placing the shotcrete (include placement in conditions when ground water is encountered).
   i. Encapsulation system for additional corrosion protection selected for the soil nails and anchorages requiring encapsulation.
4. Detailed working drawings of the method proposed for the soil nail testing that includes:
   a. All necessary drawings and details to clearly describe the proposed system of jacking support, framing, and bracing to be used during testing.
   b. Calibration data for each load cell, test jack, pressure gauge, stroke counter on the grout pump, and master gauge to be used. The calibration tests shall have been performed by an independent testing laboratory, and tests shall have been performed within 60 calendar days of the date submitted. Testing or work shall not commence until the Engineer has approved the load cell, jack, pressure gage, and master pressure gauge calibrations.
5. Certified mill test results and typical stress-strain curves along with samples from each heat, properly marked, for the soil nail steel. The typical stress-strain curve shall be obtained by approved standard practices. The guaranteed ultimate strength, yield strength, elongation, and composition shall be specified.

6-15.3(4) Preconstruction Conference

A soil nail preconstruction conference shall be held at least five working days prior to the Contractor beginning any permanent soil nail work at the site to discuss construction procedures, personnel and equipment to be used. The list of materials specified on the Record of Materials Form (ROM) for this item of work will also be discussed. Those attending shall include:

1. (representing the Contractor) The superintendent, on site supervisors, and all foremen in charge of excavating the soil face, drilling the soil nail hole, placing the soil nail and grout, placing the shotcrete facing, and tensioning and testing the soil nail.
2. (representing the Contracting Agency) The Project Engineer, key inspection personnel, and representatives from the WSDOT Construction Office and Materials Laboratory Geotechnical Services Branch.

If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved permanent soil nail installation plan, an additional conference shall be held before any additional permanent soil nail operations are performed.

6-15.3(5) Earthwork

The ground contour above the wall shall be established to its final configuration and slope as shown in the Plans prior to beginning excavation of the soil for the first row of soil nails. All excavation shall conform to Section 2-03.

The excavation shall proceed from the top down in a horizontal lift sequence with the ground level excavated no more than 3-feet below the elevation of the row of nails to be installed in that lift. The excavated vertical wall face should not be left open more than 24 hours for any reason. A lift shall not be excavated until the nail installation and reinforced shotcrete placement for the preceding lift has been completed and accepted. After a lift is excavated, the cut surface shall be cleaned of all loose materials, mud, rebound, and other foreign matter that could prevent or reduce shotcrete bond.

The accuracy of the ground cut shall be such that the required thickness of shotcrete can be placed within a tolerance of plus or minus 2-inches from the defined face of the wall, and over excavation does not damage overlying shotcrete sections by undermining or other causes.

The Contractor should review the geotechnical recommendations report prepared for this project for further information on the soil conditions at the location of each wall. Copies of the geotechnical recommendations report are available for review by prospective bidders at the location identified in the Special Provisions.

6-15.3(6) Soil Nailing

The Contractor shall not handle and transport the encapsulated soil nails until the encapsulation grout has reached sufficient strength to resist damage during handling. The Contractor shall handle the encapsulated soil nails in such a manner to prevent large deflections or distortions during handling. When handling or transporting encapsulated soil nails, the Contractor shall provide slings or other equipment necessary to prevent damage to the soil nails and the corrosion protection. The Engineer may reject any encapsulated nail which is damaged during transportation or handling. Damaged or defective encapsulation shall be repaired in accordance with the manufacturer's recommendations and as approved by the Engineer.

Soil nails shall be handled and sorted in such a manner as to avoid damage or corrosion. Prior to inserting a soil nail in the drilled hole, the Contractor and the Engineer will examine the soil nail for damage. If, in the opinion of the Engineer, the epoxy coating or bar has been damaged, the nail shall be repaired. If, in the opinion of the Engineer, the damage is beyond repair, the soil nail shall be rejected.

If, in the opinion of the Engineer, the epoxy coating can be repaired, the Contractor shall patch the coating with an Engineer approved patching material.

Nail holes shall be drilled at the locations shown in the Plans or as staked by the Engineer. The nails shall be positioned plus or minus 6-inches from the theoretical location shown in the Plans. The Contractor shall select the drilling method and the grouting pressure used for the installation of the soil nail. The drill hole shall be located so that the longitudinal axis of the drill hole and the longitudinal axis of the nail are parallel. At the point of entry the soil nail shall be installed within plus or minus three degrees of the inclination from horizontal shown in the Plans, and the nail shall be within plus or minus three degrees of a line drawn perpendicular to the face of the wall unless otherwise shown in the Plans.

Water or other liquids shall not be used to flush cuttings during drilling, but air may be used. After drilling, the nail shall be installed and fully grouted before placing the shotcrete facing. The nail shall be inserted into the drilled hole with centralizers to the desired depth in such a manner as to prevent damage to the drilled hole, sheathing or epoxy during installation. The centralizers shall provide a minimum of 0.5-inches of grout cover over the soil nail and shall be spaced no further than eight feet apart. When the soil nail cannot be completely inserted into the drilled hole without difficulty, the Contractor shall remove the nail from the drilled hole and clean or redrill the hole to permit insertion. Partially inserted soil nails shall not be driven or forced into the hole. Subsidence, or any other detrimental impact from drilling shall be cause for immediate cessation of drilling and repair of all damages in a manner approved by the Engineer at no additional cost to the Contracting Agency.

If caving conditions are encountered, no further drilling will be allowed until the Contractor selects a method to prevent ground movement. The Contractor may use temporary casing. The Contractor's method to prevent ground movement shall be approved by the Engineer. The casings for the nail holes, if used, shall be removed as the grout is being placed.

Where necessary for stability of the excavation face, a sealing layer of shotcrete may be placed before drilling is started, or the Contractor shall have the option of drilling and grouting of nails through a stabilizing berm of native soil at the face of the excavation. The stabilizing berm shall extend horizontally from the soil face and from the face of the shotcrete a minimum distance of one foot, and shall be cut down from that point at a safe slope, no steeper than 1H:1V unless approved by the Engineer. The berm shall be excavated to final grade after installation and full length grouting of the nails. Nails damaged during berm excavation shall be repaired or replaced by the Contractor, to the satisfaction of the Engineer, at no added cost to the Contracting Agency.

If sections of the wall are constructed at different times than the adjacent soil nail sections, the Contractor shall use stabilizing berms, temporary slopes, or other measures, as approved by the Engineer, to prevent sloughing or failure of the adjacent soil nail sections.

If cobbles and boulders are encountered at the soil face during excavation, the Contractor shall remove all cobbles and boulders that protrude from the soil face into the design wall section and fill the void with shotcrete. All shotcrete used to fill voids created by removal of cobbles and boulders shall be incidental to shotcrete facing.

The grout equipment shall produce a grout free of lumps and undispersed cement. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gauge near the discharge end to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used by the Contractor, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

The grout shall be injected from the lowest point of the drilled hole. The grout shall be pumped through grout tubes after insertion of the soil nail. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive ground heave.

6-15.3(7) Shotcrete Facing

Prior to placing any shotcrete on an excavated layer, the Contractor shall vertically center prefabricated drainage mat between the columns of nails as shown in the Plans. The prefabricated drainage mat shall be installed in accordance with the manufacturer's recommendations. The permeable drain side shall be placed against the exposed soil face. The prefabricated drainage mat shall be installed after each excavation lift and shall be hydraulically connected with the prefabricated drainage mat previously placed, such that the vertical flow of water is not impeded. The Contractor shall tape all joints in the prefabricated drainage mat to prevent shotcrete intrusion during shotcrete application.

The Contractor shall place steel reinforcing bars and welded wire fabric, and apply the shotcrete facing in accordance with Section 6-18 and the details shown in the Plans.

The shotcrete shall be constructed to the minimum thickness as shown in the Plans. Costs associated with additional thickness of shotcrete due to over excavation or irregularities in the cut face shall be borne by the Contractor.

Each soil nail shall be secured at the shotcrete facing with a steel plate as shown in the Plans. The plate shall be seated on a wet grout pad of a pasty consistency similar to that of mortar for brick-laying. The nut shall then be sufficiently tightened to achieve full bearing surface behind the plate. After the shotcrete and grout have had time to gain the specified strength, the nut shall be tightened with at least 100 foot-pounds of torque.

6-15.3(8) Soil Nail Testing and Acceptance

Both verification and proof testing of the nails is required. The Contractor shall supply all materials, equipment, and labor to perform the tests. The Contractor shall submit all test data to the Engineer.

The testing equipment shall include a dial gauge or vernier scale capable of measuring to 0.001-inch of the ground anchor movement. A hydraulic jack and pump shall be used to apply the test load. The movement-measuring device shall have a minimum travel equal to the theoretical elastic elongation of the total nail length plus 1-inch. The dial gauge or vernier scale shall be aligned so that its axis is within 5 degrees from the axis of the nail and shall be monitored with a reference system that is independent of the jacking system and excavation face.

The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. Each load cell, test jack and pressure gauge, grout pump stroke counter, and master gauge, shall be calibrated as specified in Section 6-15.3(3) item 4b. Additionally, the Contractor shall not use load cells, test jacks and pressure gauges, grout pump stroke counters, and master gauges, greater than 60 calendar days past their most recent calibration date, until such items are re-calibrated by an independent testing laboratory.

The pressure gauge shall be graduated in 100 psi increments or less. The pressure gauge will be used to measure the applied load. The pressure gauge shall be selected to place the maximum test load within the middle two-thirds of the range of the gauge. The ram travel of the jack shall not be less than the theoretical elastic elongation of the total length at the maximum test load plus 1-inch. The jack shall be independently supported and centered over the nail so that the nail does not carry the weight of the jack. The Contractor shall have a second calibrated jack pressure gauge at the site. Calibration data shall provide a specific reference to the jack and the pressure gauge.

The loads on the nails during the verification and proof tests shall be monitored to verify consistency of load – defined as maintaining the test load within five percent of the specified value. Verification and proof test loads less than 20,000 pounds or sustained for five minutes or less shall be monitored by the jack pressure gauge alone. Verification and proof test loads equal to or greater than 20,000 pounds and sustained for longer than five minutes shall be monitored with the assistance of an electric or hydraulic load cell. The Contractor shall provide the load cell, the readout device, and a recent calibration curve. The load cell shall be selected to place the maximum test load within the middle two-thirds of the range of the load cell. The load cell shall be mounted between the jack and the anchor plate. The stressing equipment shall be placed over the nail in such a manner that the jack bearing plates, load cell and stressing anchorage are in alignment.

Nails to be tested shall be initially grouted no closer to the excavation face than the dimension shown in the Plans. After placing the grout, the nail shall remain undisturbed until the grout has reached a strength sufficient to provide resistance during testing. Grouting to the excavation face shall be completed after successful testing has been performed. Test nails that are not part of the permanent wall may be left in the ground provided the drill holes for the nails are completely filled with grout or non-structural filler after testing.

Load testing shall be performed against a temporary bearing yoke or reaction frame that bears directly against the existing soil or the shotcrete facing. Temporary bearing pads shall be kept a minimum of 12-inches from the edges of the drilled hole unless a rigid steel plate is used to distribute the stress around the drilled hole. If a steel plate is used, it shall be a minimum of 3-feet square and of sufficient thickness that it will distribute the load evenly to the soil. Where the reaction frame bears directly against the shotcrete, the reaction frame shall be designed to prevent fracture of the shotcrete. No part of the reaction frame shall bear within 12-inches of the edge of the test nail blockout unless otherwise approved by the Engineer.

The soil nail load monitoring procedure for verification and proof test load greater than 20,000 pounds and sustained for longer than five minutes shall be as follows:

1. For each increment of load, attainment of the load shall be initially established and confirmed by the reading taken from the jack gauge.
2. Once the soil nail anchor load has been stabilized, based on the jack gauge reading, the load cell readout device shall immediately be read and recorded to establish the load cell reading to be used at this load. The load cell reading is intended only as a confirmation of a stable soil nail load, and shall not be taken as the actual load on the soil nail.
3. During the time period that the load on the soil nail is held at this load increment, the Contractor shall monitor the load cell reading. The Contractor shall adjust the jack pressure as necessary to maintain the initial load cell reading. Jack pressure adjustment for any other reason will not be allowed.
4. Soil nail elongation measurements shall be taken at each load increment as specified in Sections 6-15.3(8)A and 6-15.3(8)B.
5. Steps 1 through 4 shall be repeated at each increment of load, in accordance with the load sequence specified in Sections 6-15.3(8)A and 6-15.3(8)B.

6-15.3(8)A Verification Testing

Verification testing shall be performed on nails installed within the pattern of production nails to verify the Contractor's procedures, hole diameter, and design assumptions. No drilling or installation of production nails will be permitted in any ground/rock unit unless successful verification testing of anchors in that unit has been completed and approved by the Engineer, using the same equipment, methods, nail inclination, nail length, and hole diameter as planned for the production nails. Changes in the drilling or installation method may require additional verification testing as determined by the Engineer and shall be done at no additional expense to the Contracting Agency. Verification tests may be performed prior to excavation for the soil nail wall.

Successful verification tests are required within the limits as specified in the Special Provisions. Test nail locations within these limits shall be at locations selected by the Engineer.

The design details of the verification testing, including the system for distributing test load pressures to the excavation surface and appropriate nail bar size and reaction plate, shall be developed by the Contractor, subject to approval by the Engineer. The intent is to stress the bond between the grout and the surrounding soil/rock to at least twice the design load transfer.

The bar shall be proportioned such that the maximum stress at 200 percent of the test load does not exceed 80 percent of the yield strength of the steel. The jack shall be positioned at the beginning of the test such that unloading and repositioning of the jack during the test will not be required. The verification tests shall be made by incrementally loading the nails in accordance with the following schedule of hold time:

AL	1 minute
0.25TL	10 minutes
0.50TL	10 minutes
0.75TL	10 minutes
1.00TL	10 minutes
1.25TL	10 minutes
1.50TL	60 minutes
1.75TL	10 minutes
2.00TL	10 minutes

AL = Nail Alignment Load

TL = Nail Test Load

The test load shall be determined by the following equation = Test Load (TL) = Bond Length (BL) X Design Load Transfer (DLT).

The load shall be applied in increments of 25 percent of the test load. Each load increment shall be held for at least 10 minutes. Measurement of nail movement shall be obtained at each load increment. The load-hold period shall start as soon as the load is applied and the nail movement with respect to a fixed reference shall be measured and recorded at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes.

The Engineer will evaluate the results of each verification test and make a determination of the suitability of the test and of the Contractor's proposed production nail design and installation system. Tests that fail to meet the design criteria will require additional verification testing or an approved revision to the Contractor's proposed production nail design and installation system. If a nail fails in creep, retesting will not be allowed.

A verification tested nail with a 60 minute load hold at 1.50TL is acceptable if:

1. The nail carries the test load with a creep rate that does not exceed 0.08-inch per log cycle of time and is at a linear or decreasing creep rate.
2. The total movement at the test load exceeds 80 percent of the theoretical elastic elongation of the non-bonded length.

Furthermore, a pullout failure shall not occur for the verification test anchor at the 2.0TL maximum load. Pullout failure load is defined as the load at which attempts to increase the test load result only in continued pullout movement of the test nail without a sustainable increase in the test load.

The nails used for verification tests shall be sacrificial and shall not be used for production. The Contractor shall cut and remove the exposed end of all soil nails used for verification tests a minimum of two feet inside the finished ground line.

6-15.3(8)B Proof Testing

Proof tests shall be performed on production nails at the locations selected by the Engineer. Up to five percent of the production nails will be tested. Prior to testing, only the bond length (BL) portion of the nail shall be grouted. The Contractor shall maintain the side-wall stability of the drill hole for the non-grouted portion during the test. Once proof testing is completed, the remainder of the proof tested nail shall be grouted. The bond length shall be determined from the Nail Schedule and Test Nail Detail shown in the Plans.

Proof tests shall be performed by incrementally loading the nail in accordance with the schedule below. The anchor movement shall be measured and recorded to the nearest 0.001-inch with respect to an independent fixed reference point in the same manner as for the verification tests at the alignment load and at each increment of load. The load shall be monitored in accordance with Section 6-15.3(8). The scheduling of hold times shall be as follows:

AL	1 minute
0.25TL	5 minutes
0.50TL	5 minutes
0.75TL	5 minutes
1.00TL	5 minutes
1.25TL	5 minutes
1.50TL	10 minutes

AL = Nail Alignment Load

TL = Nail Test Load

The maximum load in a proof test shall be held for 10 minutes. The load hold period shall start as soon as the maximum load is applied and the nail movement with respect to an independent fixed reference shall be measured and recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. The nail movement between 1 minute and 10 minutes shall not exceed 0.04-inches. If the nail movement between 1 and 10 minutes exceeds 0.04-inches, the maximum load shall be held an additional 50 minutes. If the load hold is extended, the nail movement shall be recorded at 20, 30, 50, and 60 minutes. If a nail fails in creep, retesting will not be allowed.

A proof tested nail is acceptable if:

1. The nail carries the maximum load with less than 0.04-inches of movement between 1 minute and 10 minutes, unless the load hold extended to 60 minutes, in which case the nail would be acceptable if the creep rate does not exceed 0.08-inches per log cycle of time.
2. The total movement at the maximum load exceeded 80 percent of the theoretical elastic elongation of the non-bonded length.
3 The creep rate is not increasing with time during the load hold period.

Due to the requirement for a non-bonded zone for testing purposes, the Contractor shall develop an installation method which will assure the stability of the non-bonded portion of the hole during testing and will allow for the non bonded zone to be grouted against the ground after testing.

If a proof test fails, the Engineer may direct the Contractor to replace some or all of the installed production nails between the failed test and an adjacent proof test nail that has met the test criteria. The Engineer may also require additional proof testing. All additional proof tests, and all installation of additional or modified nails, shall be performed at no additional expense to the Contracting Agency.

6-15.3(9) Concrete Fascia Panels

The Contractor shall construct the concrete fascia panels in accordance with Section 6-02 and the details in the Plans. The concrete fascia panels shall be cured in accordance with the Section 6-02.3(11) requirements specified for retaining walls. The Contractor shall provide the specified surface finish as noted, and to the limits shown, in the Plans to the exterior concrete surface. When noted in the Plans, the Contractor shall apply pigmented sealer to the limits shown in the Plans.

Asphalt or cement concrete gutter shall be constructed as shown in the Plans and as specified in Section 8-04.

6-15.4 Measurement

Prefabricated drainage mat will be measured by the square yard of material furnished and installed.

Soil nails will be measured per each for each soil nail installed and accepted.

The soil nail verification testing program will not be measured but will be paid for on a lump sum basis.

Shotcrete facing and concrete fascia panel will be measured by the square foot surface area of the completed facing or fascia panel, measured to the neat lines of the facing or panel as shown in the Plans.

6-15.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Soil Nail – Epoxy Coated", per each.

"Soil Nail – Encapsulated", per each.

All costs in connection with furnishing and installing the soil nails as specified shall be included in the unit contract price per each for "Soil Nail - ___", including all drilling, grouting, centralizers, bearing plates, welded shear connectors, nuts, proof testing, and other work required for installation of each soil nail.

"Prefabricated Drainage Mat", per square yard.

"Soil Nail Verification Test", lump sum.

"Concrete Fascia Panel", per square foot.

All costs in connection with constructing the concrete fascia panels as specified shall be included in the unit contract price per square foot for "Concrete Fascia Panel", including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified).

Shotcrete facing will be paid for in accordance with Section 6-18.5.

Unless otherwise specified, all costs in connection with excavation in front of the back face of the shotcrete facing shall be included in the unit contract price per cubic yard for "Roadway Excavation" or "Roadway Excavation Incl. Haul" as specified in Section 2-03.5.

6-16 SOLDIER PILE AND SOLDIER PILE TIEBACK WALLS

6-16.1 Description

This work consists of constructing soldier pile walls and soldier pile tieback walls.

6-16.2 Materials

Materials shall meet the requirements of the following sections:

Controlled Density Fill	2-09.3(1)E
Cement	9-01
Aggregates for Portland Cement Concrete	9-03.1
Gravel Backfill	9-03.12
Premolded Joint Filler	9-04.1(2)
Welded Shear Studs	9-06.15
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Paints	9-08
Timber Lagging	9-09.2
Preservative Treatment for Timber Lagging	9-09.3(1)
Soldier Piles	9-10.5
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25
Prefabricated Drainage Mat	9-33.2(3)

Other materials required shall be as specified in the Special Provisions.

6-16.3 Construction Requirements

6-16.3(1) Quality Assurance

The steel soldier piles shall be placed so that the centerline of the pile at the top is within 1-inch of the plan location. The steel soldier pile shall be plumb, to within 0.5 percent of the length based on the total length of the pile.

Welding, repair welding, and welding inspection shall conform to the Section 6-03.3(25) requirements for welding, repair welding, and welding inspection for all other steel fabrication.

6-16.3(2) Submittals

The Contractor shall submit shop plans as specified in Section 6-03.3(7) for all structural steel, including the steel soldier piles and the permanent ground anchors to the Engineer for approval.

The Contractor shall submit the permanent ground anchor grout mix design and the procedures for placing the grout to the Engineer for approval.

The Contractor shall submit forming plans for the concrete fascia panels, as specified in Sections 6-02.3(16) and 6-02.3(17), to the Engineer for approval.

1. Where the lateral pressure from concrete placement, as specified in Section 6-02.3(17)J, is less than or equal to the design earth pressure, the Contractor may tie forms directly to the soldier piles.

2. Where the lateral pressure from concrete placement, as specified in Section 6-02.3(17)J, is greater than the design earth pressure, the Contractor shall follow one of the following procedures:
   a. Tie the forms to strongbacks behind the lagging, or use some other system that confines the pressure from concrete placement between the lagging and the form panels, in addition to the ties to the soldier piles.
   b. Reduce the rate of placing concrete to reduce the pressure from concrete placement to less than or equal to the design earth pressure in addition to the ties to the soldier piles.
   c. Follow a procedure with a combination of a. and b.
3. The Contractor shall design the forms for an appropriate rate of placing concrete so that no cold joints occur, considering the wall thickness and height, and volume of concrete to be placed.

The Contractor shall submit four copies of a shaft installation plan in accordance with Section 6-01.9 not less than 30 calendar days prior to the beginning of shaft construction. In preparing the submittal, the Contractor shall reference the available subsurface data provided in the contract test hole boring logs and the geotechnical report(s) prepared for this project. This plan shall provide at least the following information:

1. An overall construction operation sequence and the sequence of shaft construction.
2. List, description, and capacities of proposed equipment including but not limited to cranes, drills, augers, bailing buckets, final cleaning equipment, and drilling units. The narrative shall describe why the equipment was selected, and describe equipment suitability to the anticipated site and subsurface conditions. The narrative shall include a project history of the drilling equipment demonstrating the successful use of the equipment on shafts of equal or greater size in similar soil/rock conditions.
3. Details of shaft excavation methods including proposed drilling methods, methods for cleanout of the shafts, disposal plan for excavated material and drilling slurry (if applicable), and a review of method suitability to the anticipated site and subsurface conditions.
4. Details of the method(s) to be used to ensure shaft stability (i.e., prevention of caving, bottom heave, etc. using temporary casing, slurry, or other means) during excavation and concrete placement. This shall include a review of method suitability to the anticipated site and subsurface conditions. If temporary casings are proposed, casing dimensions and detailed procedures for casing installation and removal shall be provided. If slurry is proposed, detailed procedures for mixing, using, maintaining, and disposing of the slurry shall be provided. A detailed mix design, and a discussion of its suitability to the anticipated subsurface conditions shall also be provided for the proposed slurry.
5. Details of soldier pile placement including internal support bracing and centralization methods.

6. Details of concrete placement including proposed operational procedures for pumping and/or tremie methods.
7. Details of the device used to prevent unauthorized entry into a shaft excavation.
8. The method to be used to form the horizontal construction joint at the top elevation specified for concrete Class 4000P in the shaft.

Work shall not begin until the Engineer has approved the appropriate submittals in writing.

6-16.3(3) Shaft Excavation

Shafts shall be excavated to the required depth as shown in the Plans. The minimum diameter of the shaft shall be as shown in the Plans. The excavation shall be completed in a continuous operation using equipment capable of excavating through the type of material expected to be encountered.

The Contractor may use temporary telescoping casing to construct the shafts.

If the shaft excavation is stopped the shaft shall be secured by installation of a safety cover. It shall be the Contractor's responsibility to ensure the safety of the shaft and surrounding soil and the stability of the sidewalls. A temporary casing, slurry, or other methods specified in the shaft installation plan as approved by the Engineer shall be used if necessary to ensure such safety and stability.

Where caving in conditions are encountered, no further excavation will be allowed until the Contractor has implemented the method to prevent ground caving as submitted in accordance with item 4 of the Shaft Installation Plan and as approved by the Engineer.

No more than 2-inches of loose or disturbed material, for soldier piles with permanent ground anchors, nor more than 12-inches of loose or disturbed material, for soldier piles without permanent ground anchors, shall be present at the bottom of the shaft just prior to beginning concrete placement.

The excavated shaft shall be inspected and approved by the Engineer prior to proceeding with construction.

When obstructions are encountered, the Contractor shall notify the Engineer promptly. An obstruction is defined as a specific object (including, but not limited to, boulders, logs, and man made objects) encountered during the shaft excavation operation that prevents or hinders the advance of the shaft excavation. When efforts to advance past the obstruction to the design shaft tip elevation result in the rate of advance of the shaft drilling equipment being significantly reduced relative to the rate of advance for the rest of the shaft excavation, then the Contractor shall remove the obstruction under the provisions of Section 6-16.5 as supplemented in the Special Provisions. The method of removal of such obstructions, and the continuation of excavation shall be as proposed by the Contractor and approved by the Engineer.

Excavation of shafts shall not commence until a minimum of 12 hours after the shaft backfill for the adjacent shafts has been placed.

The temporary casings for the shafts shall be removed. A minimum 5 foot head of concrete shall be maintained to balance the soil and water pressure at the bottom of the casing. The casing shall be smooth.

6-16.3(4) Installing Soldier Piles

Soldier piles, if spliced, shall conform to all requirements of Section 6-05.3(6).

The prefabricated steel soldier piles shall be lowered into the drilled shafts and secured in position. Concrete cover over the soldier pile shall be 1-inch minimum.

The steel soldier piles and attachments shall be shop painted after fabrication to the limits shown in the Plans with one coat of inorganic zinc primer. Application of the one coat of primer shall be in accordance with Section 6-07. The welded shear studs may be attached before or after painting. Paint damaged by welding shear studs in place does not require repair.

6-16.3(5) Backfilling Shaft

The excavated shaft shall be backfilled with controlled density fill (CDF), lean concrete, or concrete Class 4000P, as shown in the Plans.

Placement of the shaft backfill shall commence immediately after completing the shaft excavation and receiving the Engineer's approval of the excavation. Concrete Class 4000P and lean concrete shall be placed in one continuous operation to the elevation shown in the Plans. CDF shall be placed in one continuous operation to the top of the shaft. Vibration of shaft backfill is not required.

If water is not present, the shaft backfill shall be deposited by a method that prevents segregation of aggregates. The shaft backfill shall be placed such that the free-fall is vertical down the shaft without hitting the sides of the soldier pile or the excavated shaft. The Contractor's method for depositing the shaft backfill shall have approval of the Engineer prior to the placement of the shaft backfill.

If water is present, the shaft backfill shall be deposited in accordance with Section 6-02.3(6)B.

6-16.3(6) Installing Timber Lagging and Permanent Ground Anchors

The excavation and removal of CDF and lean concrete for the lagging installation shall proceed in advance of the lagging. The bottom of the excavation in front of the wall shall be level. Excavation shall conform to Section 2-03.

For walls without permanent ground anchors, the bottom of excavation shall be not more than three feet below the bottom level of the timber lagging already installed. For walls with permanent ground anchors, the bottom of excavation shall be not more than 3-feet below tieback anchor level until all permanent ground anchors at that level are installed and stressed. Installing, stressing, and testing the permanent ground anchors shall be in accordance with Section 6-17 and the construction sequence specified in the Plans.

Unless otherwise specified, timber lagging in walls with concrete fascia panels shall be untreated. Timber lagging for all other walls shall be treated.

The lagging shall be installed from the top of the pile proceeding downward. The timber lagging shall make direct contact with the soil. Voids shall be filled with gravel backfill for walls, which shall be considered incidental to the installation of the timber lagging.

Where timber lagging and backfill are above the existing or excavated ground line, the lagging and backfill shall be placed concurrently. The backfill layers shall be placed in accordance with Section 2-03.3(14) except that all layers shall be compacted to 90 percent of maximum density.

6-16.3(7) Prefabricated Drainage Mat

For walls with concrete fascia panels, prefabricated drainage mat shall be installed full height of the concrete fascia panel and full width between soldier pile flanges, unless otherwise shown in the Plans.

The prefabricated drainage mat shall be attached to the lagging in accordance with the manufacturer's recommendations. The fabric side shall face the lagging. Splicing of the prefabricated drainage mat shall be in accordance with the manufacturer's recommendations.

The Contractor shall ensure the hydraulic connection of the prefabricated drainage mat to the previously installed material so that the vertical flow of water is not impeded.

The Contractor shall tape all joints in the prefabricated drainage mat to prevent concrete intrusion during concrete fascia panel construction.

6-16.3(8) Concrete Fascia Panel

The Contractor shall construct the concrete fascia panels as shown in the Plans, and in accordance with the forming plan as approved by the Engineer. The concrete fascia panels shall be cured in accordance with the Section 6-02.3(11) requirements specified for retaining walls.

The Contractor shall provide the specified surface finish as noted, and to the limits shown, in the Plans to the exterior concrete surface. When noted in the Plans, the Contractor shall apply pigmented sealer to the limits shown in the Plans.

Asphalt or cement concrete gutter shall be constructed as shown in the Plans.

6-16.4 Measurement

Soldier pile shaft construction will be measured by the linear foot of shaft excavated below the top of ground line for the shaft, defined as the highest existing ground point within the shaft diameter.

Furnishing soldier pile will be measured by the linear foot of pile assembly specified in the Proposal, including adjustments to the Plan quantity made in accordance with Section 1-04.4.

Timber lagging will be measured by the thousand board feet measure (MBM) installed. The quantity will be computed using the nominal thickness and width dimensions of the timber lagging members, and the center-to-center spacing of the soldier piles as the length dimension.

Prefabricated drainage mat will be measured by the square yard of material furnished and installed.

Concrete fascia panel will be measured by the square foot surface area of the completed fascia panel, measured to the neat lines of the panel as shown in the Plans.

6-16.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Shaft - ___ Diameter", per linear foot.

All costs in connection with constructing soldier pile shafts shall be included in the unit contract price per linear foot for "Shaft - ___ Diameter", including shaft excavation, temporary casing if used, CDF, lean concrete, concrete Class 4000P, and installing the soldier pile assembly.

"Furnishing Soldier Pile - ___", per linear foot.

All costs in connection with furnishing soldier pile assemblies shall be included in the unit contract price per linear foot for "Furnishing Soldier Pile - ___", including fabricating and painting the pile assemblies. Payment will be made based on the quantity specified in the Proposal unless changes are made to this quantity in accordance with Section 1-04.4, in which case the quantity specified in the Proposal will be adjusted by the amount of the change and will be paid for in accordance with Section 1-04.4.

"Timber Lagging", per MBM.

All costs in connection with furnishing and installing timber lagging shall be included in the unit contract price per MBM for "Timber Lagging", including preservative treatment when specified, and filling voids behind the lagging with gravel backfill for wall.

"Prefabricated Drainage Mat", per square yard.

"Concrete Fascia Panel", per square foot.

All costs in connection with constructing the concrete fascia panels as specified shall be included in the unit contract price per square foot for "Concrete Fascia Panel", including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified).

Unless otherwise specified, all costs in connection with non-shaft excavation, including all excavation required for placement of timber lagging, shall be included in the unit contract price per cubic yard for "Roadway Excavation" or "Roadway Excavation Incl. Haul" as specified in Section 2-03.5.

6-17 PERMANENT GROUND ANCHORS

6-17.1 Description

This work consists of constructing permanent ground anchors.

6-17.2 Materials

Materials required, including materials for permanent ground anchors, shall be as specified in the Special Provisions.

6-17.3 Construction Requirements

The Contractor shall select the ground anchor type and the installation method, and determine the bond length and anchor diameter. The Contractor shall install ground anchors that will develop the load indicated in the Plans and verified by tests specified in Sections 6-17.3(8)A, 6-17.3(8)B, and 6-17.3(8)C.

6-17.3(1) Definitions

Anchor Devices: The anchor head wedges or nuts that grip the prestressing steel.

Bearing Plate: The steel plate that evenly distributes the ground anchor force to the structure.

Bond Length: The length of the ground anchor that is bonded to the ground and transmits the tensile force to the soil or rock.

Ground Anchor: A system, referred to as a tieback or as an anchor, used to transfer tensile loads to soil or rock. A ground anchor includes all prestressing steel, anchorage devices, grout, coatings, sheathings and couplers if used.

Maintaining Consistency of Load: Maintaining the test load within five percent of the specified value.

Minimum Guaranteed Ultimate Tensile Strength (MUTS): The minimum guaranteed breaking load of the prestressing steel as defined by the specified standard.

Tendon Bond Length: The length of the tendon that is bonded to the anchor grout.

Tendon Unbonded Length: The length of the tendon that is not bonded to the anchor grout.

Total Anchor Length: The unbonded length plus the tendon bond length.

6-17.3(2) Contractor Experience Requirements

The Contractor or Subcontractor performing this work shall have installed permanent ground anchors for a minimum of three years. Prior to the beginning of construction, the Contractor shall submit a list containing at least five projects on which the Contractor has installed permanent ground anchors. A brief description of each project and a reference shall be included for each project listed. As a minimum, the reference shall include an individual's name and current phone number.

The Contractor shall assign an engineer to supervise the work with at least three years of experience in the design and construction of permanently anchored structures. The Contractor shall not use consultants or manufacturer's representatives in order to meet the requirements of this section. Drill operators and on-site supervisors shall have a minimum of one year experience installing permanent ground anchors.

Contractors or Subcontractors that are specifically prequalified in Class 36 work will be considered to have met the above experience requirements.

The Contractor shall allow up to 15 calendar days for the Engineer's review of the qualifications and staff as noted above. Work shall not be started on any anchored wall system nor materials ordered until approval of the Contractor's qualifications are given.

6-17.3(3) Submittals

The Contractor shall submit working drawings and structural design calculations in accordance with Section 6-01.9 for the ground anchor system or systems intended for use.

The Contractor shall submit a detailed description of the construction procedure proposed for use to the Engineer for approval.

The Contractor shall submit a ground anchor schedule giving:

1. Ground anchor number
2. Ground anchor design load
3. Type and size of tendon
4. Minimum total bond length
5. Minimum anchor length
6. Minimum tendon bond length
7. Minimum unbonded length

The Contractor shall submit working drawings of the ground anchor tendon and the corrosion protection system. Include details of the following:

1. Spacers and their location
2. Centralizers and their location
3. Unbonded length corrosion protection system, including the permanent rubber seal between the trumpet and the tendon unbonded length corrosion protection.
4. Bond length corrosion protection system
5. Anchorage and trumpet
6. Anchorage corrosion protection system
7. Anchors using non-restressable anchorage devices

The Contractor shall submit shop plans as specified in Section 6-03.3(7) for all structural steel, including the permanent ground anchors to the Engineer for review and approval.

The Contractor shall submit the grout mix designs and the procedures for placing the grout to the Engineer for approval. The Contractor shall also submit the methods and materials used in filling the annulus over the unbonded length of the anchor.

The Contractor shall submit five copies of detailed working drawings in accordance with Section 6-01.9 for the method proposed to be followed for the permanent ground anchor testing to the Engineer for approval prior to the tests. This shall include all necessary drawings and details to clearly describe the method proposed.

The Contractor shall submit to the Engineer calibration data for each load cell, test jack, pressure gauge and master pressure gauge to be used. The calibration tests shall have been performed by an independent testing laboratory and tests shall have been performed within 60 calendar days of the date submitted. The Engineer shall approve or reject the calibration data after receipt of the data. Testing shall not commence until the Engineer has approved the load cell, jack, pressure gauge and master pressure gauge calibrations.

Work shall not begin until the Engineer has approved the appropriate submittals in writing.

6-17.3(4) Preconstruction Conference

A permanent ground anchor preconstruction conference shall be held at least five working days prior to the Contractor beginning any permanent ground anchor work at the site to discuss construction procedures, personnel, and equipment to be used. The list of materials specified on the Record of Materials Form (ROM) for this item of work will also be discussed. Those attending shall include:

1. (representing the Contractor) The superintendent, on site supervisors, and all foremen in charge of drilling the ground anchor hole, placing the permanent ground anchor and grout, and tensioning and testing the permanent ground anchor.
2. (representing the Contracting Agency) The Project Engineer, key inspection personnel, and representatives from the WSDOT Construction Office and Materials Laboratory Geotechnical Services Branch.

If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved permanent ground anchor installation plan, an additional conference shall be held before any additional permanent ground anchor operations are performed.

6-17.3(5) Tendon Fabrication

The tendons can be either shop or field fabricated. The tendon shall be fabricated as shown in the approved shop plans.

The Contractor shall select the type of tendon to be used. The tendon shall be sized so the design load does not exceed 60 percent of the minimum guaranteed ultimate tensile strength of the tendon. In addition, the tendon shall be sized so the maximum test load does not exceed 80 percent of the minimum guaranteed ultimate tensile strength of the tendon.

The Contractor shall be responsible for determining the bond length and tendon bond length necessary to develop the design load indicated in the Plans in accordance with Sections 6-17.3(8)A, 6-17.3(8)B, and 6-17.3(8)C. The minimum bond length shall be ten feet in rock and 15-feet in soil.

When the Plans require the tendon bond length to be encapsulated, the tendon bond length portion of the tendon shall be corrosion protected by encapsulating the tendon in a grout-filled PE or PVC tube as specified in Section 6-17.2 as supplemented in the Special Provisions. The tendons can be grouted inside the encapsulation prior to inserting the tendon in the drill hole or after the tendon has been placed in the drill hole. Expansive admixtures can be mixed with the encapsulation grout if the tendon is grouted inside the encapsulation while outside the drill hole. The tendon shall be centralized within the bond length encapsulation with a minimum of 0.20-inches of grout cover. Spacers shall be used along the tendon bond length of multi-element tendons to separate the elements of the tendon so the prestressing steel will bond to the encapsulation grout.

Centralizers shall be used to provide a minimum of 0.5-inches of grout cover over the tendon bond length encapsulation. Centralizers shall be securely attached to the encapsulation and the center-to-center spacing shall not exceed ten feet. In addition, the upper centralizer shall be located a maximum of five feet from the top of the tendon bond length and the lower centralizer shall be located a maximum of one foot from the bottom of the tendon bond length.

The centralizer shall be able to support the tendon in the drill hole and position the tendon so a minimum of 0.5-inches of grout cover is provided and shall permit free flow of grout.

Centralizers are not required on encapsulated, pressure-injected ground anchor tendons if the ground anchor is installed in coarse grained soils (more than 50 percent of the soil larger than the number 200 sieve) using grouting pressures greater than 150 psi.

Centralizers are not required on encapsulated, hollow-stem-augered ground anchor tendons if the ground anchor is grouted through and the hole is maintained full of a stiff grout (eight-inch slump or less) during extraction of the auger.

The minimum unbonded length of the tendon shall be the greater of 15-feet or that indicated in the Plans.

Corrosion protection of the unbonded length shall be provided by a sheath completely filled with corrosion inhibiting grease or grout. If grease is used under the sheath, provisions shall be made to prevent the grease from escaping at the ends of the sheath. The grease shall completely coat the tendon and fill the voids between the tendon and the sheath. The working drawings shall show how the Contractor will provide a transition between the tendon bond length and the unbonded tendon length corrosion protection.

If the sheath is not fabricated from a smooth tube, a separate bond breaker shall be provided. The bond breaker shall prevent the tendon from bonding to the anchor grout surrounding the tendon unbonded length.

The total anchor length shall not be less than that indicated in the Plans or the approved working drawings.

Anchorage devices shall be capable of developing 95 percent of the minimum guaranteed ultimate tensile strength of the prestressing steel tendon. The anchorage devices shall conform to the static strength requirements of Section 3.1 of the Post Tensioning Institute "Specification for Unbonded Single Strand Tendons, First Edition - 1993".

Non-restressable anchorage devices may be used except where indicated in the Plans.

Restressable anchorages shall be provided on those ground anchors that require reloading. The post-tensioning supplier shall provide a restressable anchorage compatible with the post-tensioning system provided.

The bearing plates shall be sized so the bending stresses in the plate do not exceed the yield strength of the steel when a load equal to 95 percent of the minimum guaranteed ultimate tensile strength of the tendon is applied, and the average bearing stress on the concrete does not exceed that recommended in Section 3.1.3 of the Post Tensioning Institute, "Specification For Unbonded Single Strand Tendons, First Edition - 1993".

The trumpet shall have an inside diameter equal to or larger than the hole in the bearing plate. The trumpet shall be long enough to accommodate movements of the structure during testing and stressing. For strand tendons with encapsulation over the unbonded length, the trumpet shall be long enough to enable the tendon to make a transition from the diameter or the tendon in the unbonded length to the diameter of the tendon at the anchor head without damaging the encapsulation. Trumpets filled with corrosion-inhibiting grease shall have a permanent rubber seal, as approved by the Engineer, provided between the trumpet and the tendon unbonded length corrosion protection. Trumpets filled with grout shall have a temporary seal provided between the trumpet and the tendon unbonded length corrosion protection or the trumpet shall overlap the tendon unbonded length corrosion protection.

6-17.3(6) Tendon Storage And Handling

Tendons shall be handled and stored in such a manner as to avoid damage or corrosion. Damage to the prestressing steel as a result of abrasions, cut, nicks, welds and weld splatter will be cause for rejection by the Engineer. The prestressing steel shall be protected if welding is to be performed in the vicinity. Grounding of welding leads to the prestressing steel is forbidden. Prestressing steel shall be protected from dirt, rust, and deleterious substances. A light coating of rust on the steel is acceptable. If heavy corrosion or pitting is noted, the Engineer will reject the affected tendons.

The Contractor shall use care in handling and storing the tendons at the site. Prior to inserting a tendon in the drill hole, the Contractor and the Engineer will examine the tendon for damage to the encapsulation and the sheathing. If, in the opinion of the Engineer, the encapsulation is damaged, the Contractor shall repair the encapsulation in accordance with the tendon supplier's recommendations and as approved by the Engineer. If, in the opinion of the Engineer, the smooth sheathing has been damaged, the Contractor shall repair it with ultra high molecular weight polyethylene (PE) tape. The tape shall be spiral wound around the tendon so as to completely seal the damaged area. The pitch of the spiral shall ensure a double thickness at all points.

6-17.3(7) Installing Permanent Ground Anchors

The Contractor shall select the drilling method, the grouting procedure, and the grouting pressure used for the installation of the ground anchor.

When caving conditions are encountered, no further drilling will be allowed until the Contractor selects a method to prevent ground movement. The Contractor may use a temporary casing. The Contractor's method to prevent ground movement shall be approved by the Engineer. The casings for the anchor holes, if used, shall be removed. The drill hole shall be located so the longitudinal axis of the drill hole and the longitudinal axis of the tendon are parallel. The ground anchor shall not be drilled in a location that requires the tendon to be bent in order to enable the bearing plate to be connected to the supported structure. At the point of entry the ground anchor shall be installed within plus or minus three degrees of the inclination from horizontal shown in the Plans or the approved working drawings. The ground anchors shall not extend beyond the right of way limits.

The tendon shall be inserted into the drill hole to the desired depth. When the tendon cannot be completely inserted without difficulty, the Contractor shall remove the tendon from the drill hole and clean or redrill the hole to permit insertion. Partially inserted tendons shall not be driven or forced into the hole.

The Contractor shall use a grout conforming to Section 6-17.2 as supplemented in the Special Provisions.

The grout equipment shall produce a grout free of lumps and undispersed cement. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gauge near the discharge end to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used by the Contractor, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

The grout shall be injected from the lowest point of the drill hole. The grout may be pumped through grout tubes, casing, or drill rods. The grout can be placed before or after insertion of the tendon. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive heave in soils or fracturing of rock formations.

After grouting, the tendon shall not be loaded for a minimum of 3 days.

No grout shall be placed above the top of the bond length during the time the bond length grout is placed. The grout at the top of the drill hole shall not contact the back of the structure or the bottom of the trumpet. Except as otherwise noted, only nonstructural filler shall be placed above the bond length grout prior to testing and acceptance of the anchor. The Contractor may place structural grout above the bond length grout prior to testing and acceptance of the anchor subject to the following conditions:

1. The anchor unbonded length shall be increased by eight feet minimum.
2. The grout in the unbonded zone shall not be placed by pressure grouting methods.

The corrosion protection surrounding the unbonded length of the tendon shall extend up beyond the bottom seal of the trumpet or one foot into the trumpet if no trumpet seal is provided. If the protection does not extend beyond the seal or sufficiently far enough into the trumpet, the Contractor shall extend the corrosion protection or lengthen the trumpet.

The corrosion protection surrounding the no load zone length of the tendon shown in the Plans shall not contact the bearing plate or the anchor head during testing and stressing. If the protection is too long, the Contractor shall trim the corrosion protection to prevent contact.

The bearing plate and anchor head shall be placed so the axis of the tendon and the drill hole are both perpendicular to the bearing plate within plus or minus three degrees and the axis of the tendon passes through the center of the bearing plate.

The trumpet shall be completely filled with corrosion inhibiting grease or grout. Trumpet grease can be placed anytime during construction. Trumpet grout shall be placed after the ground anchor has been tested. The Contractor shall demonstrate to the Engineer that the procedure selected by the Contractor for placement of either grease or grout produces a completely filled trumpet.

All anchorages permanently exposed to the atmosphere shall be covered with a corrosion inhibiting grease-filled or grout-filled cover. The Contractor shall demonstrate to the Engineer that the procedures selected by the Contractor for placement of either grease or grout produces a completely filled cover. If the Plans require restressable anchorages, corrosion inhibiting grease shall be used to fill the anchorage cover and trumpet.

6-17.3(8) Testing And Stressing

Each ground anchor shall be tested. The test load shall be simultaneously applied to the entire tendon. Stressing of single elements of multi-element tendons will not be permitted. The Engineer will record test data.

The testing equipment shall consist of a dial gauge or vernier scale capable of measuring to 0.001-inches shall be used to measure the ground anchor movement. The movement-measuring device shall have a minimum travel equal to the theoretical elastic elongation of the total anchor length plus 1-inch. The dial gauge or vernier scale shall be aligned so that its axis is within 5 degrees from the axis of the tieback. A hydraulic jack and pump shall be used to apply the test load. The jack and pressure gauge shall be calibrated by an independent testing laboratory as a unit. Each load cell, test jack and pressure gauge, and master pressure gauge, shall be calibrated as specified in Section 6-17.3(3). Additionally, the Contractor shall not use load cells, test jacks and pressure gauges, and master pressure gauges, greater than 60 calendar days past their most recent calibration date, until such items are re-calibrated by an independent testing laboratory.

The pressure gauge shall be graduated in 100-psi increments or less. The pressure gauge will be used to measure the applied load. The pressure gauge shall be selected to place the maximum test load within the middle two-thirds of the range of the gauge. The ram travel of the jack shall not be less than the theoretical elastic elongation of the total anchor length at the maximum test load plus one inch. The jack shall be independently supported and centered over the tendon so that the tendon does not carry the weight of the jack. The Contractor shall have a second calibrated jack pressure gauge at the site. Calibration data shall provide a specific reference to the jack and the pressure gauge.

The loads on the tiebacks during the performance and verification tests shall be monitored to verify consistency of load as defined in Section 6-17.3(1). Performance test loads, and verification test loads when specified in the Special Provisions, sustained for five minutes or less, and all proof test leads, shall be monitored by the jack pressure gauge alone. Performance test loads, and verification test loads when specified in the Special Provisions, sustained for longer than five minutes shall be monitored with the assistance of an electric or hydraulic load cell. The Contractor shall provide the load cell and a readout device. The load cell shall be mounted between the jack and the anchor plate. The load cell shall be selected to place the maximum test load within the middle two-thirds of the range of the load cell. The stressing equipment shall be placed over the ground anchor tendon in such a manner that the jack, bearing plates, load cell and stressing anchorage are in alignment.

The permanent ground anchor load monitoring procedure for performance test loads, and verification test loads when specified in the Special Provisions, sustained for longer than five minutes shall be as follows:

1. For each increment of load, attainment of the load shall be initially established and confirmed by the reading taken from the jack gauge.
2. Once the permanent ground anchor load has been stabilized, based on the jack gauge reading, the load cell readout device shall immediately be read and recorded to establish the load cell reading to be used at this load. The load cell reading is intended only as a confirmation of a stable permanent ground anchor load, and shall not be taken as the actual load on the permanent ground anchor.
3. During the time period that the load on the permanent ground anchor is held at this load increment, the Contractor shall monitor the load cell reading. The Contractor shall adjust the jack pressure as necessary to maintain the initial load cell reading. Jack pressure adjustment for any other reason will not be allowed.
4. Permanent ground anchor elongation measurements shall be taken at each load increment as specified in Sections 6-17.3(8)A and 6-17.3(8)B.
5. Steps 1 through 4 shall be repeated at each increment of load, in accordance with the load sequence specified in Sections 6-17.3(8)A and 6-17.3(8)B.

6-17.3(8)A Verification Testing

Verification tests will be required only when specified in the Special Provisions.

6-17.3(8)B Performance Testing

Performance tests shall be done in accordance with the following procedures. Five percent of the ground anchors or a minimum of three ground anchors, whichever is greater, shall be performance tested. The Engineer shall select the ground anchors to be performance tested. The first production anchor shall be performance tested.

The performance test shall be made by incrementally loading and unloading the ground anchor in accordance with the following schedule, consistent with the design method (Load Resistance Factor Design - LRFD or Load Factor Design - LFD) specified in the permanent ground anchor general notes in the Plans. The load shall be raised from one increment to another immediately after a deflection reading.

Performance Test Schedule

Load Resistance Factor Design Method (LRFD)	**Load Factor Design Method (LFD)**
Load	Load
AL	AL
0.25FDL	0.25DL
AL	AL
0.25FDL	0.25DL
0.50FDL	0.50DL
AL	AL
0.25FDL	0.25DL
0.50FDL	0.50DL
0.75FDL	0.75DL
AL	AL
0.25FDL	0.25DL
0.50FDL	0.50DL
0.75FDL	0.75DL
1.00FDL	1.00DL
AL	AL
Jack to lock-off load	0.25DL
	0.50DL
	0.75DL
	1.00DL
	1.25DL
	AL
	0.25DL
	0.50DL
	0.75DL
	1.00DL
	1.25DL
	1.33DL
	Jack to lock-off load

Where: AL - is the alignment load
DL - is the design load
FDL - is the factored design load.

The maximum test load in a performance test shall be held for ten minutes. The load-hold period shall start as soon as the maximum test load is applied and the anchor movement, with respect to a fixed reference, shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, and 10 minutes. If the anchor movement between one minute and ten minutes exceeds 0.04-inches, the maximum test load shall be held for an additional 50 minutes. If the load hold is extended, the anchor movement shall be recorded at 15 minutes, 20, 25, 30, 45, and 60 minutes. If an anchor fails in creep, retesting will not be allowed. All anchors not performance tested shall be proof tested.

6-17.3(8)C Proof Testing

Proof tests shall be performed by incrementally loading the ground anchor in accordance with the following schedule, consistent with the design method (Load Resistance Factor Design - LRFD or Load Factor Design - LFD) specified in the permanent ground anchor general notes in the Plans. The load shall be raised from one increment to another immediately after a deflection reading. The anchor movement shall be measured and recorded to the nearest 0.001-inches with respect to an independent fixed reference point at the alignment load and at each increment of load. The load shall be monitored with a pressure gauge. At load increments other than the maximum test load, the load shall be held just long enough to obtain the movement reading.

Proof Test Schedule

Load Resistance Factor Design Method (LRFD)	Load Factor Design Method (LFD)
Load	Load
AL	AL
0.25FDL	0.25DL
0.50FDL	0.50DL
0.75FDL	0.75DL
1.00FDL	1.00DL
Jack to lock-off load	1.25DL
	1.33DL
	Jack to lock-off load

Where: AL - is the alignment load
DL - is the design load
FDL - is the factored design load

The maximum test load in a proof test shall be held for ten minutes. The load-hold period shall start as soon as the maximum test load is applied and the anchor movement with respect to a fixed reference shall be measured and recorded at 1 minute, 2, 3, 4, 5, 6, and 10 minutes. If the anchor movement between one minute and ten minutes exceeds 0.04-inches, the maximum test load shall be held of an additional 50 minutes. If the load hold is extended, the anchor movements shall be recorded at 15 minutes, 20, 25, 30, 45, and 60 minutes. If an anchor fails in creep, retesting will not be allowed.

6-17.3(9) Permanent Ground Anchor Acceptance Criteria

A performance or proof tested ground anchor with a ten minute load hold is acceptable if the:

1. Ground anchor carries the maximum test load with less than 0.04-inches of movement between one minute and ten minutes; and
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon unbonded length.

A verification, performance or proof tested ground anchor with a 60-minute load hold is acceptable if the:

1. Ground anchor carries the maximum test load with a creep rate that does not exceed 0.08-inches/log cycle of time and is a linear or decreasing creep rate.
2. Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the tendon unbonded length.

If the total movement of the ground anchors at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the tendon unbonded length, the Contractor shall replace the ground anchor at no additional cost to the Contracting Agency. Retesting of a ground anchor will not be allowed.

Ground anchors that have a creep rate greater than 0.08-inches/log cycle of time can be incorporated in the finished work at a load equal to one-half its failure load. The failure load is the load carried by the anchor after the load has been allowed to stabilize for ten minutes.

When a ground anchor fails, the Contractor shall modify the design, the construction procedures, or both. These modifications may include, but are not limited to, installing replacement ground anchors, modifying the installation methods, increasing the bond length or changing the ground anchor type. Any modification that requires changes to the structure shall have prior approval of the Engineer. Any modifications of design or construction procedures shall be at the Contractor’s expense.

Upon completion of the test, the load shall be adjusted to the lock-off load indicated in the Plans and transferred to the anchorage device. The ground anchor may be completely unloaded prior to lock-off. After transferring the load and prior to removing the jack a lift-off reading shall be made. The lift-off reading shall be within ten percent of the specified lock-off load.

If the load is not within ten percent of the specified lock-off load, the anchorage shall be reset and another lift-off reading shall be made. This process shall be repeated until the desired lock-off load is obtained.

6-17.4 Measurement

Permanent ground anchors will be measured per each for each permanent ground anchor installed and accepted.

Permanent ground anchor performance tests will be measured per each for each anchor performance tested.

The permanent ground anchor verification testing program will not be measured but will be paid for on a lump sum basis.

6-17.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Permanent Ground Anchor", per each.

All costs in connection with furnishing and installing permanent ground anchors shall be included in the unit contract price per each for "Permanent Ground Anchor", including proof testing of the installed anchor as specified.

"Permanent Ground Anchor Performance Test", per each.

"Permanent Ground Anchor Verification Test", lump sum.

6-18 SHOTCRETE FACING

6-18.1 Description

This work consists of constructing shotcrete facing as shown in the Plans. Shotcrete constructed as concrete slope protection shall be constructed in accordance with Section 8-16.

6-18.2 Materials

Materials shall meet the requirements of the following sections:

Cement	9-01
Aggregates for Portland Cement Concrete	9-03.1
Premolded Joint Filler	9-04.1(2)
Steel Reinforcing Bar	9-07.2
Epoxy-Coated Steel Reinforcing Bar	9-07.3
Concrete Curing Materials and Admixtures	9-23
Fly Ash	9-23.9
Water	9-25

Other materials required, including materials for shotcrete, shall be as specified in the Special Provisions.

6-18.3 Construction Requirements

6-18.3(1) Submittals

The Contractor shall submit the following information to the Engineer at least 14 calendar days prior to beginning construction of the shotcrete facing:

1. The shotcrete mix design with compressive strength test results.
2. Method and equipment used to finish and cure the shotcrete facing.
3. Documentation of the experience of the nozzle operators in applying shotcrete.

The Contractor shall not begin construction of the shotcrete facing until receiving the Engineer's approval of the above submittals.

6-18.3(2) Mix Design

Shotcrete shall be proportioned to produce a 4,000 psi compressive strength at 28 days. The Contractor shall submit the shotcrete mix design, proposed method of placement, and evidence that the proposed design and placement method will produce the desired compressive strength at 28 days, to the Engineer at least 14 calendar days prior to the anticipated beginning of shotcrete placement. Shotcrete placement will not be allowed until the Engineer has approved the mix design and method of placement.

Admixture shall be used only after receiving permission from the Engineer. If admixtures are used to entrain air, to reduce water-cement ratio, to retard or accelerate setting time, or to accelerate the development of strength, the admixtures shall be used at the rate specified by the manufacturer and approved by the Engineer.

6-18.3(3) Testing

The Contractor shall make shotcrete test panels for evaluation of shotcrete quality, strength, and aesthetics. Both preproduction and production test panels, shall be prepared. All cores obtained for the purpose of shotcrete strength testing shall have the following minimum dimensions:

a. The core diameter shall be at least 3 times the maximum aggregate size, but not less than 2-inches.

b. The core height shall be a minimum of 1.5 times the core diameter.

The Contractor shall remove at least three cores from each 36-inch by 36-inch shotcrete test panel in accordance with AASHTO T 24. Cores removed from the panel shall be immediately wrapped in wet burlap and sealed in a plastic bag. Cores shall be clearly marked to identify from where they were taken and whether they are for pre-production or production testing. If for production testing, the section of the wall represented by the cores shall be clearly marked on the cores. Cores shall be delivered to the Engineer within 2 hours of coring. The remainder of the panels shall remain the property of the Contractor.

6-18.3(3)A Pre-production Testing

At least one 36-inch by 36-inch panel for each mix design shall be prepared for evaluation and testing of the shotcrete quality and strength. One 48-inch by 48-inch qualification panel shall be prepared for evaluation and approval of the proposed method for shotcrete installation, finishing, and curing. Both the 36-inch and the 48-inch panels shall be constructed using the same methods and initial curing proposed to construct the shotcrete facing, except that the 36-inch panel shall not include wire reinforcement. The 36-inch panel shall be constructed to the minimum thickness necessary to obtain the required core samples. The 48-inch panel shall be constructed to the same thickness as proposed for the production facing. Production shotcrete work shall not begin until satisfactory test results are obtained and the panels are approved by the Engineer.

6-18.3(3)B Production Testing

The Contractor shall make at least one 36-inch by 36-inch panel for each section of facing shot. A section is defined as one day's placement. The production panels shall be constructed using the same methods and initial curing used to construct the shotcrete wall, but without wire reinforcement. The panels shall be constructed to the minimum thickness necessary to obtain the required core samples. If the production shotcrete is found to be unsuitable based on the results of the test panels, the section(s) of the wall represented by the test panel(s) shall be repaired or replaced to the satisfaction of the Engineer at no additional cost to the Contracting Agency.

6-18.3(4) Qualifications of Contractor's Personnel

All nozzle operators shall have had at least one year of experience in the application of shotcrete. Each nozzle operator will be qualified, by the Engineer, to place shotcrete, after successfully completing one test panel for each shooting position and surface type which will be encountered.

Qualification will be based on a visual inspection of the shotcrete density, void structure, and finished appearance along with a minimum 7-day compressive strength of 2,500 psi determined from the average test results from two cores taken from each test panel.

The Contractor shall notify the Engineer not less than 2 days prior to the shooting of a qualification panel. The mix design for the shotcrete shall be the same as that slated for the wall being shot.

Shotcrete shall be placed only by personnel qualified by the Engineer.

If shotcrete finish Alternative B or C is specified, evidence shall be provided that all shotcrete crew members have completed at least three projects in the last five years where such finishing, or sculpturing and texturing of shotcrete was performed.

6-18.3(5) Placing Wire Reinforcement

Reinforcement of the shotcrete shall be placed as shown in the Plans. The wire reinforcement shall be securely fastened to the steel reinforcing bars so that it will be 1 to 1.5-inches from the face of the shotcrete at all locations, unless otherwise shown in the Plans. Wire reinforcement shall be lapped 1.5 squares in all directions, unless otherwise shown in the Plans.

6-18.3(6) Alignment Control

The Contractor shall install non-corroding alignment wires and thickness control pins to establish thickness and plane surface. The Contractor shall install alignment wires at corners and offsets not established by formwork. The Contractor shall ensure that the alignment wires are tight, true to line, and placed to allow further tightening. The Contractor shall remove the alignment wires after facing construction is complete.

6-18.3(7) Shotcrete Application

A clean, dry supply of compressed air sufficient for maintaining adequate nozzle velocity for all parts for the work and for simultaneous operation of a blow pipe for cleaning away rebound shall be maintained at all times. Thickness, method of support, air pressure, and rate of placement of shotcrete shall be controlled to prevent sagging or sloughing of freshly applied shotcrete.

The shotcrete shall be applied from the lower part of the area upwards. Surfaces to be shot shall be damp, but free of standing water.

The nozzles shall be held at an angle approximately perpendicular to the working face and at a distance that will keep rebound at a minimum and compaction will be maximized. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. If, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the work until a steady flow resumes.

Surface defects shall be repaired as soon as possible after initial placement of the shotcrete. All shotcrete which lacks uniformity; which exhibits segregation, honeycombing, or lamination; or which contains any dry patches, slugs, voids, or sand pockets, shall be removed and replaced with fresh shotcrete by the Contractor, to the satisfaction of the Engineer at no cost to the Contracting Agency.

Construction joints in the shotcrete shall be uniformly tapered over a minimum distance of twice the thickness of the shotcrete layer. The surface of the joints shall be cleaned and thoroughly wetted before adjacent shotcrete is placed. Shotcrete shall be placed in a manner that provides a finish with uniform texture and color across the construction joint.

The shotcrete shall be cured by applying a clear curing compound in accordance with Section 9-23.2. The curing compound shall be applied immediately after final gunning. The air in contact with shotcrete surfaces shall be maintained at temperatures above 50F for a minimum of 7 days. Curing compounds shall not be used on any surfaces against which additional shotcrete or other cementitious finishing materials are to be bonded unless positive measures such as sandblasting, are taken to completely remove the curing compounds prior to the application of such additional materials.

If field inspection or testing, by the Engineer, indicates that any shotcrete produced, fails to meet the requirements, the Contractor shall immediately modify procedures, equipment, or system, as necessary, and as approved by the Engineer to produce specification material. All substandard shotcrete already placed shall be repaired by the Contractor, to the satisfaction of the Engineer, at no additional cost to the Contracting Agency. Such repairs may include removal and replacement of all affected materials.

6-18.3(8) Shotcrete Finishing

When the shotcrete facing is an interim coating to be covered by a subsequent shotcrete coating or a cast-in-place concrete fascia later under the same contract, the Contractor shall strike off the surface of the shotcrete facing with a roughened surface as specified in Section 6-02.3(12). The grooves of the roughened surface shall be either vertical or horizontal.

When the shotcrete facing provides the finished exposed final surface, the shotcrete face shall be finished using the alternative aesthetic treatment shown in the Plans. The alternatives are as follows:

Alternative A

After the surface has taken its initial set (crumbling slightly when cut), the surface shall be broom finished to secure a uniform surface texture.

Alternative B

Shotcrete shall be applied in a thickness a fraction beyond the alignment wires and forms. The shotcrete shall stiffen to the point where the surface does not pull or crack when screeded with a rod or trowel. Excess material shall be trimmed, sliced, or scraped to true lines and grade. Alignment wires shall be removed and the surface shall receive a steel trowel finish, leaving a smooth uniform texture and color. Once the shotcrete has cured, pigmented sealer shall be applied to the shotcrete face. The shotcrete surface shall be completed to within a tolerance of ½-inch of true line and grade.

Alternative C

Shotcrete shall be hand-sculptured, colored, and textured to simulate the relief, jointing, and texture of the natural backdrop surrounding the facing. The ends and base of the facing shall transition in appearance as appropriate to more nearly match the color and texture of the adjoining roadway fill slopes. This may be achieved by broadcasting fine and coarse aggregates, rocks, and other native materials into the final surface of the shotcrete while it is still wet, allowing sufficient embedment into the shotcrete to become a permanent part of the surface.

6-18.4 Measurement

Shotcrete facing will be measured by the square foot surface area of the completed facing measured to the neat lines of the facing as shown in the Plans.

6-18.5 Payment

Payment will be made in accordance with Section 1-04.1 for each of the following bid items when they are included in the proposal:

"Shotcrete Facing", per square foot.

All costs in connection with constructing shotcrete facing as specified shall be included in the unit contract price per square foot for "Shotcrete Facing" including all steel reinforcing bars, premolded joint filler, polyethylene bond breaker strip, joint sealant, PVC pipe for weep holes, exterior surface finish, and pigmented sealer (when specified).